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This Book

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TO MY PARENTS
GAVE ME THE EDUCATION UPON WHICH IT IS BASED

TO MY WIFE
FOR HER LOVING SYMPATHY, ENCOURAGEMENT
AND ASSISTANCE

TO ORLANDO W. NORCROSS
OF WORCESTER, MASS.
OSE SUPERIOR PRACTICAL KNOWLEDGE OF ALL THAT
PERTAINS TO BUILDING HAS GIVEN ME A MORE
INTELLIGENT AND PRACTICAL VIEW OF THE
SCIENCE OF CONSTRUCTION THAN I
SHOULD OTHERWISE HAVE
OBTAINED *

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[illegible], Professor of Engineering, [illegible] references is largely increased and the page-numbers of such references are almost always added; many tables and diagrams which in the former editions read lengthwise of the page have been reset or reengraved to read across the page for greater convenience; the number of illustrations has been largely increased, many old cuts reused have been reengraved, and some diagrams printed with lines of different colors to make the demonstrations clearer; a descriptive caption has been added to every illustration; the abbreviations Chap. I, Chap. II, etc., have been printed with each page-caption of the left-hand pages, thus avoiding the necessity of referring to the Table of Contents to locate any particular chapter.

The Editor-in-Chief decided to change some of the unit stresses, especially those for the different woods, and in some cases to recommend more conservative values, and he believes that results based upon such stresses conform to the best engineering practice. This change necessitated the revision of many tables and problems throughout the book which had to be entirely recalculated. Numerous practical problems with complete solutions have been added. The derivation of many of the formulas used has been explained, either in the body of the text or in extended foot-notes, for those who wish to understand as well as to use such formulas, and numerous cross-references accompanying them enable the reader to use the Pocket-Book as a textbook for certain parts of the mechanics of materials as well as a handbook for office work. The tables of the properties of structural shapes, of safe loads for columns, beams and girders, etc., have been revised and numerous new tables added. The Editor has found that it is the consensus of opinion among architects that the insertion of these tables is a great convenience and for their ordinary office work condenses into one handy volume much of the essential data of several manufacturers' handbooks.

The difficulty of securing a unity of treatment and of avoiding repetitions and contradictions in a book of reference the data of which covers so many subjects and is written by so many contributors has been fully realized; but it is believed that in these respects the New Kidder is reasonably successful and will meet with the approval of all who use it.

Acknowledgments and thanks are due the Associate Editors for their hearty cooperation and generous contributions of the time and labor taken from their professional work. Acknowledgment is made, also, of the valuable assistance of all others who have furnished new or revised old data, and of many helpful suggestions from Mrs. F. E. Kidder and from the publishers.

The Editor-in-Chief expresses the hope that for the architects and builders of this country the new Pocket-Book will continue to be, as Mr. Kidder expressed it in his preface to the first edition in 1884, "a compendium of practical facts, rules and tables presented in a form as convenient for application as possible, and as reliable as our present knowledge will permit;" and also, in its present extension and fuller development, a work which will lead to a still clearer understanding of the essential principles of sound architectural construction.

THOMAS NOLAN.

PHILADELPHIA, September, 1915.

PREFACE TO FOURTEENTH EDITION

It is now nearly twenty years since the author, then quite a young man, completed the first edition of this work, which, although containing but 586 pages, had required about three years for its preparation. At that time the author thought he had covered all of those practical details relating to the planning and construction of buildings, with which the architect was concerned, tolerably well, and it would appear as though the purchasers of the book thought so too, but as the years have come and gone, so many and such great improvements have taken place in the building world, so many articles invented, new methods of construction developed, higher standards established, that the present edition, although containing nearly three times as many pages, is perhaps not more complete, for the times, than was the first edition.

When preparing the first edition, it was the aim of the author to give to architects and builders a handbook which should be, in its field, as useful and reliable as Trautwine's had been to civil engineers; and with that object constantly in view, the book has been revised from time to time to meet the changed conditions in building construction and equipment.

About three years ago it was thought, by the publishers and the author, that a thorough and complete revision of the book should be undertaken, and although the re-writing of a work of this character, even with the thirteenth edition to work from, involved many months of close and constant application, the utilization of those hours which one ordinarily takes for recreation, and at the best more or less interruption to his regular business, and consequent reduction in income, the writer undertook to prepare a work of a still wider scope, and which should be thoroughly up-to-date in every particular, or at least as far as is practicable, in a work requiring a period of three years in its preparation, and from that time to this he has spared no labor or expense to make the book as useful and complete as he possibly could, without making it too bulky.

In this revision the author has had in view:

1st. A reference-book which should contain some information on every subject (except design) likely to come before an architect, structural engineer, draughtsman, or master-builder, including data for estimating the approximate cost.

2d. To as thoroughly cover the subject of architectural engineering as is practicable in a handbook.

3d. To present all information in as simple and convenient a form for immediate application as is consistent with accuracy. To this end a great many new tables, arranged and computed by the author, have been inserted.

At the time the first edition was written, the term "Architectural Engineering" had not been used in its present application, and the term "Structural Engineering," when used, referred almost exclusively to bridge work.

To-day, structural and architectural engineers are concerned almost exclusively with building construction, and their work is more closely allied to that of the architect than to that of the civil engineer; hence the author has had in mind the needs of the structural engineer and draughtsman as well as those of the architect and builder, and the book should be of nearly equal value to both.

Where it was impossible, for lack of space, to go extensively into any subject, references to other books or sources of information have been given, so that in this way the book may serve as a general index to the many lines of work, materials, and manufactured products entering into the planning, construction, and equipment of buildings.

To attain the objects in view, it has been necessary to add considerably to the number of pages, but as experience has shown that the book is used principally at the desk or draughting-table, and is seldom carried in the pocket, it is believed that the convenience of having everything in one book will more than offset any disadvantage resulting from increase in bulk.

Nearly the entire book has been re-written, and great pains have been taken to furnish reliable data. A large number of experts in various lines have assisted the author, as is manifest by the foot-notes and references. To all of such, and to the many authors of technical works, and to the publishers of technical journals, who have kindly consented to the use of cuts and data, the author takes pleasure in acknowledging his indebtedness. Also to Mr. E. S. Hand, of New York, who, for many years, has rendered material assistance in collecting data along the line of manufactured products.

The names and addresses of manufacturers have been given solely for the convenience of the users of the book, and not for any pecuniary considerations; in fact, if money considerations had solely appealed to the writer, this book would never have been re-written, because a technical work of this character can never adequately compensate, in money, for the time, labor, and thought required in its preparation. The many words of appreciation which have come to the author from hundreds of those who have found the book useful have been a great stimulus to further increase its usefulness.

As in the former prefaces, the author requests that any one discovering errors in the work or who may have any suggestions looking to the further improvement of the book, will communicate the same to him, that the book may be made as complete and reliable as possible.

Finally, the author desires to acknowledge his indebtedness to the publishers, who have heartily seconded his efforts in every particular, and who have spared no pains or expense to make a perfect handbook.

F. E. KIDDER.

DENVER, COLO., July 18th, 1904.

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ARCHITECTURAL SOCIETIES 1696
LIST OF VALUABLE BOOKS FOR ARCHITECTS 1703
PERIODICALS DEVOTED TO ARCHITECTURE 1710
GLOSSARY 1713
ARCHITECTURAL TERMS USED IN LAW 1768

201
2021)3581
2021
20227)156026
141589
14437

Cube Root. To extract the cube root of a number, point off the
from right to left into periods of three figures each, and, if there is a
commence at the decimal point and point off into periods, going both wa

Ascertain the highest root of the first period, and place it to the righ
number, as in long division; cube the root thus found and subtract fi
first period; to the remainder annex the next period; square the root
found, multiply by three and annex two ciphers for the trial divisor. F
many times this trial divisor is contained in the dividend and write th
in the root.

Add together the trial divisor, three times the product of the first f
the root by the second with one cipher annexed, and the square of the
figure in the root; multiply the sum by the last figure in the root, and
from the dividend; to the remainder annex the next period and proceed as

When the trial divisor is greater than the dividend, write a cipher in t
annex the next period to the dividend and proceed as before.

Example. Required, the cube root of 493039 or $\sqrt[3]{493039}$

493039(79, the cube root

7 × 7 × 7 = 343

7 × 7 × 3 = 14700 | 150039
7 × 9 × 3 = 1890
9 × 9 = 81
16671 | 150039

Example. Required, the cube root of 403583.419 or $\sqrt[3]{403583.419}$

403583.419(73.9, the cube root

Scripture and Ancient Measures and Weights

Scripture Long Measures

	Inches		Feet	Inches
Digit	= 0.912	Cubit	= 1	9.888
Palm	= 3.648	Fathom	= 7	3.552
Span	= 10.944			

Egyptian Long Measures

Nahud cubit = 1 foot 5.71 inches Royal cubit = 1 foot 8.66 inches

Grecian Long Measures

	Feet	Inches		Feet	Inches
Digit	=	0.7554	Stadium	= 604	4.5
Pous (foot)	= 1	0.0875	Mile	= 4 835	
Cubit	= 1	1.5984⅜			

Jewish Long Measures

Cubit	= 1.824 feet	Mile	= 7 296 feet
Sabbath-day's journey	= 3 648 feet	Day's journey	= 33.164 miles

Roman Long Measures

	Inches		Feet	Inches
Digit	= 0.72575	Cubit	= 1	5.406
Uncia (inch)	= 0.967	Passus	= 4	10.02
Pes (foot)	= 11.604	Mille (millarium)	= 4 842	

Roman Weight

Ancient libbra = 0.7094 pound

Miscellaneous

	Feet		Feet
Arabian foot	= 1.095	Hebrew foot	= 1.212
Babylonian foot	= 1.140	Hebrew cubit	= 1.817
Egyptian finger	= 0.06145	Hebrew sacred cubit	= 2.002

Metric Conversion Tables. This and the following table from Molesworth's Metrical Tables will be found of great convenience in figuring plans to be executed in Mexico and other countries using the metric system.

Feet Converted into Meters

Feet	0	1	2	3	4
0	....	0.304794	0.609589	0.914383	1.21918
10	3.047945	3.35274	3.65753	3.96233	4.26712
20	6.095890	6.40068	6.70548	7.01027	7.31507
30	9.143835	9.44863	9.75342	10.0582	10.3630
40	12.19178	12.4966	12.8014	13.1062	13.4110
50	15.23972	15.5445	15.8493	16.1541	16.4589
60	18.28767	18.5925	18.8973	19.2020	19.5068
70	21.33561	21.6404	21.9452	22.2500	22.5548
80	24.38356	24.6884	24.9931	25.2979	25.6027
90	27.43150	27.7363	28.0411	28.3459	28.6507

An ARC of a circle is any part of its circumference.

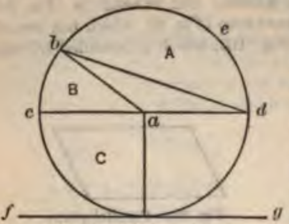

Fig. 19. Circle and Parts

A CHORD is any straight line joini points of the circumference, as *bd*, F

A SEGMENT is a portion of the included between the arc and its ch *A*, Fig. 19.

A SECTOR is the space included b an arc and two radii drawn to i tremities, as *B*, Fig. 19. In the fig is a radius, *cd* a diameter and *db* a SUBTENDING the arc *bed*. A TANGE right line which in passing a curve t without cutting it, as *fg*, Fig. 19.

Volumes

A PRISM is a volume whose ends are equal and parallel polygons and sides are parallelograms.

A prism is TRIANGULAR, RECTANGULAR, etc., according as its ends a ANGLES, RECTANGLES, etc.

A CUBE is a rectangular prism all of whose sides are squares.

A CYLINDER is a volume of uniform diameter, bounded by a curved and two equal and opposite parallel circles.

A PYRAMID is a volume whose base is a polygon and whose sides are tr meeting in a point called the VERTEX. A pyramid is triangular, quadra etc., according as its base is a triangle, quadrilateral, etc.

A CONE is a volume whose base is a circle, from which the remaining surface tapers uniformly to a point or vertex (Fig. 20).

A CONIC SECTION is the plane figure made by a plane cutting a cone.

An ELLIPSE is the section of a cone cut by a plane passing obliquely through both sides, as at *ab*, Fig. 21.

A PARABOLA is a section of a cone cut by a plane parallel to its side, as at *cd*.

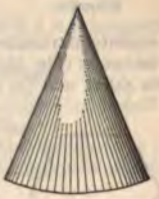

Fig. 20.
Cone

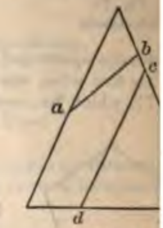

Fig. 21.
Cone with Sectio

A HYPERBOLA is a section of a cone cut by a plane making a greate with the base than that made by the side of the cone, as at *eh*.

In the ellipse, the TRANSVERSE AXIS, or LONG DIAMETER, is the longe that can be drawn in it. The CONJUGATE AXIS, or SHORT DIAMETER, is drawn through the center at right-angles to the long diameter.

A FRUSTUM OF A PYRAMID or CONE is that which remains after cutting upper part of it by a plane parallel to the base.

A SPHERE is a volume bounded by a curved surface, all points of whi equidistant from a point within, called the center.

Mensuration treats of the measurement of lines, surfaces and volum

Rules

To compute the area of a square, a rectangle, a rhombus or a rhomboid

Rule. Multiply the length by the breadth or height. Thus, in Fig or 24, the area = $ab \times bc$.

Areas and Circumferences of Circles (Continued)

Advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
30.0	706.8583	94.2478	35.0	962.1128	109.9557	40.0	1256.6371	125.6637
.1	711.5786	94.5619	.1	967.6184	110.2699	.1	1262.9281	125.9779
.2	716.3145	94.8761	.2	973.1397	110.5841	.2	1269.2348	126.2920
.3	721.0662	95.1903	.3	978.6768	110.8982	.3	1275.5573	126.6062
.4	725.8336	95.5044	.4	984.2296	111.2124	.4	1281.8955	126.9203
.5	730.6167	95.8186	.5	989.7980	111.5265	.5	1288.2493	127.2345
.6	735.4154	96.1327	.6	995.3822	111.8407	.6	1294.6189	127.5487
.7	740.2299	96.4469	.7	1000.9821	112.1549	.7	1301.0042	127.8628
.8	745.0601	96.7611	.8	1006.5977	112.4690	.8	1307.4052	128.1770
.9	749.9060	97.0752	.9	1012.2290	112.7832	.9	1313.8219	128.4911
31.0	754.7676	97.3894	36.0	1017.8760	113.0973	41.0	1320.2543	128.8053
.1	759.6450	97.7035	.1	1023.5387	113.4115	.1	1326.7024	129.1195
.2	764.5380	98.0177	.2	1029.2172	113.7257	.2	1333.1663	129.4336
.3	769.4467	98.3319	.3	1034.9113	114.0398	.3	1339.6458	129.7478
.4	774.3712	98.6460	.4	1040.6212	114.3540	.4	1346.1410	130.0619
.5	779.3113	98.9602	.5	1046.3467	114.6681	.5	1352.6520	130.3761
.6	784.2672	99.2743	.6	1052.0880	114.9823	.6	1359.1786	130.6903
.7	789.2388	99.5885	.7	1057.8449	115.2965	.7	1365.7210	131.0044
.8	794.2260	99.9026	.8	1063.6176	115.6106	.8	1372.2791	131.3186
.9	799.2290	100.2168	.9	1069.4060	115.9248	.9	1378.8529	131.6327
32.0	804.2477	100.5310	37.0	1075.2101	116.2389	42.0	1385.4424	131.9469
.1	809.2821	100.8451	.1	1081.0299	116.5531	.1	1392.0476	132.2611
.2	814.3322	101.1593	.2	1086.8654	116.8672	.2	1398.6685	132.5752
.3	819.3980	101.4734	.3	1092.7166	117.1814	.3	1405.3051	132.8894
.4	824.4796	101.7876	.4	1098.5835	117.4956	.4	1411.9574	133.2035
.5	829.5768	102.1018	.5	1104.4662	117.8097	.5	1418.6254	133.5177
.6	834.6898	102.4159	.6	1110.3645	118.1239	.6	1425.3092	133.8318
.7	839.8185	102.7301	.7	1116.2786	118.4380	.7	1432.0086	134.1460
.8	844.9628	103.0442	.8	1122.2083	118.7522	.8	1438.7238	134.4602
.9	850.1229	103.3584	.9	1128.1538	119.0664	.9	1445.4546	134.7743
33.0	855.2986	103.6726	38.0	1134.1149	119.3805	43.0	1452.2012	135.0885
.1	860.4902	103.9867	.1	1140.0918	119.6947	.1	1458.9635	135.4026
.2	865.6973	104.3009	.2	1146.0844	120.0088	.2	1465.7415	135.7168
.3	870.9202	104.6150	.3	1152.0927	120.3230	.3	1472.5352	136.0310
.4	876.1588	104.9292	.4	1158.1167	120.6372	.4	1479.3446	136.3451
.5	881.4131	105.2434	.5	1164.1564	120.9513	.5	1486.1697	136.6593
.6	886.6831	105.5575	.6	1170.2118	121.2655	.6	1493.0105	136.9734
.7	891.9688	105.8717	.7	1176.2830	121.5796	.7	1499.8670	137.2876
.8	897.2703	106.1858	.8	1182.3698	121.8938	.8	1506.7393	137.6018
.9	902.5874	106.5000	.9	1188.4724	122.2080	.9	1513.6272	137.9159
34.0	907.9203	106.8142	39.0	1194.5906	122.5221	44.0	1520.5308	138.2301
.1	913.2688	107.1283	.1	1200.7246	122.8363	.1	1527.4502	138.5442
.2	918.6331	107.4425	.2	1206.8742	123.1504	.2	1534.3853	138.8584
.3	924.0131	107.7566	.3	1213.0396	123.4646	.3	1541.3360	139.1726
.4	929.4088	108.0708	.4	1219.2207	123.7788	.4	1548.3025	139.4867
.5	934.8202	108.3849	.5	1225.4175	124.0929	.5	1555.247	139.8009
.6	940.2473	108.6991	.6	1231.6300	124.4071	.6	1562.2826	140.1150
.7	945.6901	109.0133	.7	1237.8582	124.7212	.7	1569.2962	140.4292
.8	951.1486	109.3274	.8	1244.1021	125.0354	.8	1576.3255	140.7434
.9	956.6228	109.6416	.9	1250.3617	125.3495	.9	1583.3706	141.0575

first one at *c*. Draw from *a* a line through *c*, and it will make with *ab* an angle of 60°.

Problem 7. From a given point, A, on a given line, AE, to draw a line making an angle 45° with the given line (Fig. 56).

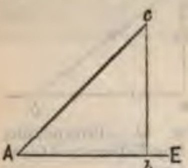

Fig. 56. Angle of 45°

Measure off from *A*, on *AE*, any distance, *Ab*, and at *b* draw a line perpendicular to *AE*. Measure off on this perpendicular *bc* equal to *Ab* and draw a line from *A* through *c*. This line *Ac* will make an angle of 45° with *AE*.

Problem 8. From any point, A, on a given line, to draw a line which will make any desired angle with the given line (Fig. 57).

To solve this problem the tables of chords on pages 81 to 89 are used. Find in the table the length of chord to a radius 1, for the given angle. Then take any radius, as large as convenient and describe an arc of a circle *bc*, with *A* as a center. Multiply the chord of the angle, found in the table, by the length of the radius *Ab*, and with the product as a new radius and with *b* as a center, describe a short arc cutting *bc* in *d*. Draw a line from *A* through *d* and it will make the required angle with *DE*.

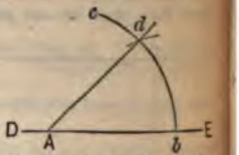

Fig. 57. Line Making Any Angle with Given Line

Example. Draw a line from *A* on *DE*, making an angle of 44° 40′ with *DE* (Fig. 57).

Solution. The largest convenient radius for the arc is 8 in. With *A* as a center and 8 in as a radius, describe the arc *bc*. In the table of chords, the chord for an angle or arc of 44° 40′ to a radius 1 is 0.76. Multiplying this by 8 in, the length of the new radius is 6.08 in; and with this as radius and with *b* as a center, describe an arc cutting *bc* in *d*. *Ad* will be the line required.

Problem 8a. To lay off a given angle approximately, by means of an ordinary two-foot rule.

Tables of Angles Corresponding to Openings of a Two-Foot Rule *

In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.
¼	1	12	...	11	22	4½	21	37	...	32	3	8¾	42	46
..	1	48	2½	11	58	...	22	13	6¾	32	40	...	43	24
½	2	24	...	12	34	¾	22	50	...	33	17	9	44	3
..	3	00	¾	13	10	...	23	27	7	33	54	...	44	42
¾	3	36	...	13	46	5	24	3	...	34	33	¼	45	21
..	4	11	3	14	22	...	24	39	¼	35	10	...	45	59
1	4	47	...	14	58	¼	25	16	...	35	47	½	46	38
..	5	23	¼	15	34	...	25	53	½	36	25	...	47	17
¼	5	58	...	16	10	½	26	30	...	37	3	¾	47	56
..	6	34	½	16	46	...	27	7	¾	37	41	...	48	35
½	7	10	...	17	22	¾	27	44	...	38	19	10	49	15
..	7	46	¾	17	59	...	28	21	8	38	57	...	49	54
¾	8	22	...	18	35	6	28	58	...	39	35	¼	50	34
...	8	58	4	19	12	...	29	35	¼	40	13	...	51	13
2	9	34	...	19	48	¼	30	11	...	40	51	½	51	53
...	10	10	¼	20	24	...	30	49	½	41	29	...	52	33
¼	10	46	...	21	00	½	31	26	...	42	7	...	....	

* Trautwine.

Fix two pins in the axis *AB* at *F* and *F'* and loop upon them a thread, or cord equal in length, when fastened to the pins, to *AB*, so as, when stretched as per dotted line *FDF'*, it will just reach to the extremity *D* of the short axis. Place a pencil-point inside the chord, as at *E*, and move the pencil along, keeping the cord stretched tight. The pencil-point will trace the ellipse required.

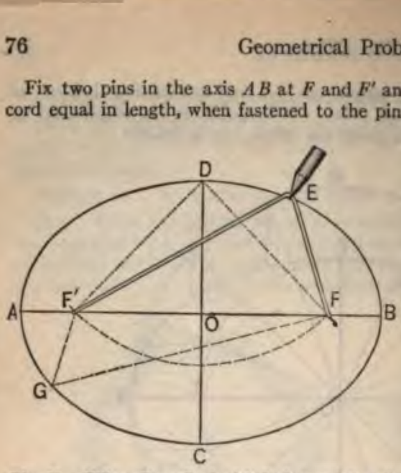

Fig. 83. Ellipse Described with String and Pencil

Problem 32. To draw a tangent to an ellipse at a given point on the curve (Fig. 84).

Let it be required to draw a tangent at the point *E* on the ellipse shown. First determine the foci *F* and *F'* as in the third method for describing an ellipse, and from *E* draw lines *EF* and *EF'*. Prolong *EF'* to *a*, so that *Ea* equals *EF*. Bisect the angle *aEF* by describing arcs from *a* and *F* as centers, as shown at *b*, and through *b* draw a line through *E*. This line is the tangent required. If it is required to draw a line normal to the curve at *E*, as, for instance, the joint of an elliptical arch, bisect the angle *FEF'*, and draw the bisecting line through *E*, and it will be the normal to the curve and the proper line at that point for the joint of an elliptical arch.

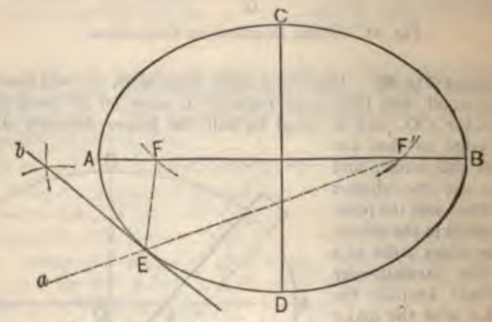

Fig. 84. Tangent Drawn to Point on Ellipse

Problem 33. To draw a tangent to an ellipse from a given point outside of the curve (Fig. 85).

From the given point *T* as a center, and with a radius equal to the distance to the nearer focus *F*, describe an arc of a circle. From *F'* as a center, and with a radius equal to the length of the longer axis of the ellipse, describe ar cutting the circle just described at *a* and *b*. Draw lines from *F'* to *a* and cutting the ellipse at *E* and *G*. Draw lines from *T* through *E* and *G* and will be the tangents required.

2. Explanation of the Notation or Symbols used in Part

Besides the usual mathematical signs and characters in general use, lowing abbreviations and symbols are frequently used:

A	area of cross-section; also, a constant used in Chapter XVI to $1/16$ the safe unit fiber-stress;
$a, b, c, \ldots m$, etc.,	known or given distances;
b	breadth, as of beams;
C	coefficient of strength;
c	normal distance from neutral axis of cross-section of beam distant fiber in same;
d	diameter, as of rivets; exterior diameter; depth, as of beams;
d_1	interior diameter;
E	modulus of elasticity;
E_s, E_c	modulus of elasticity for concrete and steel respectively (inforced concrete);
e	total deformation or change in length, as in a bar;
F	shearing-modulus of elasticity;
f	maximum deflection for a beam;
h	distance between parallel axes for moments of inertia;
I	moment of inertia about a line;
I/c	section-modulus or section-factor;
J	polar moment of inertia;
J'	polar moment of inertia of bolts about shaft-axis;
K	total elastic resistance of a bar; resilience, work; also, a factor stant used in formulas for reinforced concrete;
l	length; span of a beam;
M	bending moment;
$M_{\max}$	maximum bending moment;
M_1, M_2	bending moments at supports;
M_r or SI/c	moment of resistance;
n	number of loads, spans, etc.;
P	external force; concentrated load;
P_1, P_2, P_3, etc.,	concentrated loads on beams;
p	pitch of rivets; eccentricity of load on column; ratio of cross of steel to cross-section of beam (reinforced concrete);
r	radius of curvature; radius; radius of gyration; ratio of E_s to E_c for concrete (reinforced concrete);
R_1, R_2, R_3, etc.,	reactions at the supports of a beam;
S	unit stress, with subscripts t, c and s for unit stress in tensio pression and shear, respectively;
S_b	buckling resistance in webs of steel beams;
S_h	horizontal unit shearing-stress in beams;
S_e	elastic limit;
S_f	modulus of rupture, or computed flexural strength;
t_1, t_2, etc.,	thicknesses;
V	vertical shear;
W	weight of a bar or beam; total uniform load on beam (may weight of beam);
wl	total uniform load on a beam (may include weight of beam);
w	weight of a cubic unit of material; uniform load on beam, pe unit of length;

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[*] See, also, page 3 of Part I.

1. Definitions of Some of the Terms Used in the M Materials *

Terms Used in Architectural Engineering. The follow quently occur in discussions of the principles of architectural e an understanding of their meaning is essential.

Mechanics is the branch of physics that treats of the phenor the action of forces on material bodies.

Applied Mechanics treats of the laws of mechanics as appli tion in the useful arts, as in beams, trusses, arches, etc.

Mechanics of Materials treats of the effects of forces in c in the size and shape of bodies.

Rest is the relation that exists between two points when tl joining them does not change in length or direction. A body is a to a point when any point in the body is at rest relatively to the point.

Motion is the relation that exists between two points when tl joining them changes in length or direction, or in both. A bo tively to a point when any point in the body moves relatively to tioned point.

Force is that which changes, or tends to change, the state of of the body acted upon. It is a cause regarding the essential n we are ignorant. In the mechanics of materials we do not deal v of forces, but only with the laws of their action.

Equilibrium is that condition of a body in which the forces balance or neutralize each other; or, it is that condition of a f which the resultant of the force-system is zero.

Statics is the branch of Mechanics that treats of the conditions It is divided into:

(1) Statics of rigid bodies.

(2) Statics of practically incompressible fluids.

are three conditions of equilibrium in a structure.
l forces acting upon the whole structure must balance each
es are:
it of the structure;
it carries;
rd supporting forces, reactions or resistances under or around

acting upon each piece of the structure must balance each
es are, for each piece:
it of the piece;
it carries;
ances or reactions at its joints.
acting upon each of the parts into which any piece may be
rided must balance each other.

of a Structure requires the fulfilment of conditions (1) and
ility of the structure to resist the DISPLACEMENT of any of its

of a Piece or Member consists in the fulfilment of condi-
e ability of a piece to resist BREAKING.

of a Piece or Member consists in the ability of a piece to

Structures is divided into two parts:
treats of strength and stiffness, dealing only with single
lly known as the STRENGTH OF MATERIALS or the MECHANICS
ore defined.
treats of stability, dealing with the structures themselves.

ternal force that resists a change in shape or size caused by
When the applied external forces reach certain intensities
es hold them in equilibrium.

of a Stress is measured by the UNIT STRESS. (See Unit
ENSITY OF THE STRESS per square inch on any normal surface
tal stress divided by the area of the section in square inches.
ft long and 2 in square has a load of 8 000 lb pulling in the
gth, the stress on any normal section of the bar is 8 000 lb;
of the stress per square inch is 8 000 lb/4 sq in = 2 000 lb per

When a solid body is acted upon by an external force an
ace in the volume and shape of the body, and this alteration
MATION. In the case of the bar given above, the deformation
t the bar stretches under its load.

Strength is the highest unit stress a piece of material can
he unit stress at or just before rupture.

Unit Stress is the ultimate stress divided by the factor of

d is the load that a piece can support without exceeding the
sses.

e term STRAIN is now synonymous with the term DEFORMATION. On
ency to confuse the terms STRAIN and STRESS the term DEFORMATION,
ange in shape and the term STRAIN is omitted in all discussions in the

beam. As the factors of safety thus vary for different kinds of st
materials, the proper factors for the different kinds of stresses and con
given in considering the resistance of the various materials to tho
under varying conditions.

The Unit Stress is the stress on a unit of section-area, and is usually
in pounds per square inch. (See Intensity of Stress.)

Dead Loads and Live Loads. The term DEAD LOAD means a l
applied and increased gradually and that finally remains constant, s
weight of a structure itself.

The term LIVE LOAD means a load that is applied suddenly and ca
tions, such as a train traveling over a railway bridge. It has been
experience that the effect of a live load on a beam or other piece
has twice the destructive tendency of a dead load of the same ma
intensity. Hence a piece of material designed to carry a live load sl
a factor of safety twice as large as one designed to carry a dead
load due to a crowd of people walking on a floor is usually considered
an effect which is a mean between that of a dead load and a live l
suitable factor of safety is adopted accordingly. In municipal ordi
laws relating to the allowable loads for floors, the loads to be suppor
floors, exclusive of their inherent construction and stationary fixture
erally referred to as the LIVE LOADS no matter of what they may co
the term does not have the exact significance given to it by many
and as explained in the paragraph above.

The Modulus of Rupture or Computed Flexural Strength is
of the UNIT FIBER-STRESS S, computed from the flexure-formula
when a beam is ruptured under a transverse load. Its value is int
between the ultimate tensile and compressive strengths of a material.

The Elastic Limit is that unit stress at which the deformation of
material begins to increase in a faster ratio than the applied loa
sometimes called the ELASTIC STRENGTH.

The Modulus of Elasticity or Coefficient of Elasticity. In
on physics this is often called YOUNG'S MODULUS. It we take a bar of

Shear is the stress that resists the tendency of two equal parallel ing in opposite directions to cause two adjoining planes of a body to on the other.

Torsion is the stress that resists the tendency of forces to TWIST a

Combined Stresses. Parts of structures are often acted upon by external forces which develop stresses of different character, such as flexure and compression, flexure and tension, flexure and torsion, shear compression or tension, torsion and compression, etc.

all available information relative thereto and, if necessary, should mak and tests so as to secure reliable information on which to base his desig foundation. The first step is, therefore, a detailed and exhaustive stu site to determine the characteristics of the foundation-bed on which t ture is to rest.

3. Geological Considerations

Character of the Foundation-Bed. A knowledge of geology is of assistance in many cases in making a proper estimate of the charact foundation-bed. While it is not proposed in the limits of this chapter t any general geological discussion the following notes may be of value in the architect to determine whether any given deposits can be relied affording a stable foundation-bed. Broadly speaking, as the locatio building may be in any part of the world, so the materials encount belong to any one of the many geological formations forming the surf earth. For practical purposes, however, the materials met with ar divided into ROCK, or materials other than rock, roughly defined as E

4. Composition and Classification of Rocks

Composition of Rocks. Rocks, and the earthy deposits derived fr are composed of various minerals of which many hundred kinds ar each varying from the others in some particular of chemical composit of crystallization or other characteristic. A rock or an earthy deposit sist almost entirely of a single mineral, but it is usually composed distinct minerals or of mixtures of minerals. The principal classes forming minerals are:

(1) **The Silica Minerals,** composed of silica (SiO_2) in different forms

(2) **Silicates** or combinations of silica, with various metallic bases;

(3) **Calcareous Minerals** composed of calcite or carbonate of lime and its combinations.

(1) **Silica Minerals** are different forms of the oxide of silicon, SILICA. In the crystalline state silica is known as

Quartz. This is the most abundant of all minerals. Owing to its

, an important constituent of granite and other igneous rocks, is a alumina with potash, soda or lime. When exposed to the action of owly decomposes, forming silicate of alumina, the base of clay. The tion of granite results in the formation of clay and crystals of quartz The mica is very slowly affected and the quartz is practically un-

The various mica minerals are silicates of alumina, with potash and tituents. All varieties are soft and split into thin elastic plates. icles of mica are frequently found in sand.

nde and Augite are silicates of lime, magnesia, iron and alumina and uent occurrence.

Talc and Soapstone Travertine are hydrated silicates formed from tes by a chemical change in which a certain amount of water is These minerals are soft and have a SOAPY FEEL. Special care should n building foundations on rock of this character to guard against any the foundation-bed or between parts of the foundation-bed.

careous Minerals. The following are the principal calcareous miner-

($CaCO_3$), carbonate of lime, when pure and crystallized, is known as PAR. It is soluble in water containing CO_2. Calcite in varying de- urity forms limestone and marbles. As a result of its solubility d voids are frequently found in limestone.

e is a carbonate of lime and magnesia. It forms the so-called DOLO- STONES, which are less soluble than the calcite limestone.

, **Lypsum, Alabaster, Anhydrite, Aragonite and Apatite** are other and tant lime-minerals.

cation of Rocks. Rocks are classified not only according to the f which they are composed, but also according to the way in which been formed, as:

ous Rocks, which have solidified from a molten condition;

imentary Rocks, which have been formed under water by mechani- re or by cementation due to chemical or organic proceesses;

amorphic or Plutonic Rocks, which have changed from their original as igneous or sedimentary rocks.

eous or Plutonic Rocks are not truly stratified. They may be rystalline or glassy in texture. GRANITE, SYENITE, BASALT, TRAP, etc., les. LAVA, PUMICE and OBSIDIAN are volcanic products, as are also posits of mud and ash. With the exception of volcanic ash and mud, s rocks are enduring and are not liable to present any unforeseen weak- undation-beds.

imentary Rocks are composed of sand, clay and other materials rom the breaking down of the original igneous rocks. These materials sited in horizontal beds generally by settling from water, and the con- into rock was generally affected under water by chemical, mechanical action. The resultant rock-masses are stratified as a result of their t materials having been deposited in layers. As sand and clay are abundant products of rock-decomposition, so the sedimentary rocks frequently SILICEOUS (sandy) or ARGILLACEOUS (clayey).

ne is composed of grains of sand cemented together by silica, oxides

are no fossils or other indications of the organic origin of the calcareous
Admixtures of sand, clay, or other impurities may make it difficult to dis
certain limestones from sandstones or shales.

Dolomite is a limestone containing a high percentage of magnesia.

Hydraulic Limestone is a limestone containing clay.

Chalk is a soft limestone composed of the fine shells of minute marin
isms. In general, the purer the limestone the more soluble it is and th
the danger from fissures or caverns due to the action of water.

(3) **Metamorphic or Plutonic Rocks** are rocks which have bee
from sedimentary or igneous rocks by heat, compression, or moistu
alone or in combination. Thus by heat from a nearby intrusion of mo
limestone is changed into a crystalline marble. The general effect
MORPHISM is to produce a hard or durable rock.

Quartzite, a metamorphosed sandstone, is a crystalline rock of great
and durability.

Slate is a hard dense rock, sometimes with a well-defined tendency to
thin plates. It has been formed by metamorphic action from clayey s
is generally durable, but liable to slide along planes which are somet
allel to the cleavage, or along seams which are not parallel to the cleav

Gneiss is a "laminated metamorphic rock that usually correspon
alogically to some one of the plutonic types." * There are many vari
classified in accordance with the igneous rocks to which they most nea
spond in composition. Some varieties resemble granite, but the lam
striped aspect is generally characteristic. They are generally com
durable.

Schists are similar to gneiss but are more finely foliated or striped.
SCHIST there are layers or foliations composed of fine grains or plates
Mica-schists are liable to decomposition and it frequently happens tha
tions have to be carried to great depths through decomposed rock of
acter before solid rock is encountered. The material resulting from th
position of this rock contains fine grains of mica and other fine mat
when wet, acts as QUICKSAND.

Rock as a Foundation. All rock, if sound and not liable to sli es, the crystals of feldspar changing chemically and forming the silicate of aluminum known as CLAY, while the crystals of quartz, rnblende, being more resistant to chemical action, retain their chem- y but become detached particles of SAND.

hanical Agencies. By the MECHANICAL AGENCIES, such as the action oving water or ice, fragments of rock are detached from the ledge of originally formed part and are subsequently transported, by the laciers or streams, or by the wave-action in bodies of water. The etween the materials thus roughly thrown about breaks up the rock- o smaller and smaller pieces without altering the composition of the ial.

Water. As flowing water more readily transports small particles ones, the larger pieces of rock move intermittently during periods of ood and are deposited as soon as the velocity of the water falls; while particles are held in suspension longer and, as the velocity of the ls, are deposited in the order of their size, the largest first. The r courses of streams and rivers in mountainous regions constantly roll together the materials in their rocky beds, the heavy masses being wly. The attrition between the fragments forms GRAVEL and SAND washed down stream to be deposited, as the current slackens, first as RAVEL, then as SAND-BARS, and finally, in the slow-moving lower EDS OF SILT and ALLUVIUM.

s and Glacial Deposits. The action of glaciers is similar to the ac- eams. Glacial deposits, the so-called GLACIAL DRIFTS, are composed lay, gravel and boulders but, in general, there is a noticeable differ- en glacial deposits and deposits made by rivers or streams. In glacial he boulders frequently exhibit groovings or scratches on their faces dges and surfaces of the boulders are generally sharp, so that a ay appear as if it had been recently fractured. They rarely exhibit h, water-worn and rounded surfaces found on boulders formed by on. Moreover, the glacial boulders may be found singly, or unas- ith other boulders in a deposit of sand or gravel. The deposit differs er-deposit in that there is no classification as to size; the boulders may

tions and forms BEACHES, while the larger rock-masses remain near
of origin in BARS and REEFS.

Beds of Sand, Gravel and Boulders deposited by the action of
the SHORES OF SEAS OR LAKES are not necessarily constant in character
should be made to determine the character of the material underly
BEACH-FORMATIONS. In large river-valleys where the general form
composed of silt or other fine material little reliance should be place
occurrence of BEDS OF GRAVEL, even if such beds extend over large area
should be made to determine that such beds are not underlain by l
worthy materials. Where tributary streams discharge into large val
may deposit BARS OF SAND, GRAVEL and BOULDERS on top of the silt
other materials formerly deposited by the main river. (See page
general topographical conditions should serve as an indication of dang
cases.

Results of Chemical and Mechanical Action. As a result of
going brief description of the agencies at work it may be seen that
and STREAM-ACTION alike tend to disrupt rock-masses and to produce
gravel, sand and finer materials. The ultimate result of the comb
CHEMICAL ACTION and MECHANICAL ACTION is to reduce the hardest ro
finest sand, the most impalpable clays, silts and muds; and the ACTION
WAVE and MOVING WATER is to classify such materials in deposits of
uniform size.

6. Materials Composing Foundation-Beds

Classification and Definitions. The following list includes the
which are most frequently encountered, with their definitions.

Rock (solid rock, bed-rock, or ledge). Undisturbed rock-masses f
undisturbed part of the original rock-formation.

Decayed Rock (rotten rock). Sand, clays and other materials
from the disintegration of rock-masses, lacking the coherent qua
occupying the space formerly occupied by the original rock.

Loose Rock. Rock-masses detached from the ledge of which they

The material resulting from the decomposition and hydration of feld-
cks, being hydrated silicate of alumina, generally mixed with powdered
juartz and other materals.

Pan. Any strongly coherent mixture of clay or other cementing
vith sand, gravel, or boulders.

A finely divided earthy material deposited from running water.

Finely divided earthy material generally containing vegetable matter
sited from still or slowly moving water.

Loosely used to describe any earthy material.

Earthy material capable of supporting vegetable life and generally
material containing decayed vegetable or animal matter.

. Earthy material containing a large proportion of humus or vegetable

Earthy material containing a proportion of vegetable matter.

Compressed and partially carbonized vegetable matter.

. Characteristics of the Materials of Foundation-Beds

Rock, or, as it is locally known, BED-ROCK, or LEDGE, is proverbially
undation. The harder rocks, such as granite, trap, slate, sandstone,
, etc., are all capable of carrying the load of any ordinary structure.
r rocks, among which may be classed the shales, shaley slates and
arley limestones and clay stones, should not be loaded with more than
er sq ft unless they are tested for greater loads. In all cases where
ns are to be placed on what is supposed to be solid rock, care should be
determine whether or not the supposed solid consists of a detached
nd, also, in case the bedding-planes of the rock are inclined, if there is
om a slip of the layer forming the foundation-bed. (See pages 139 and
side-slope locations.)

ed Rock. Certain igneous or metamorphic rocks such as granites,
tc., frequently disintegrate, forming so-called ROTTEN ROCK or DECAYED
'he decayed rock is generally found in conformity with the ledge of
originally formed a part. It may retain the stratification, color and
of the solid rock, but as a result of the disintegrating effect of water
agents, it has lost the solid character of the original rock. When
th a hammer it does not give the characteristic ringing sound of solid
may be fairly compact and hard, or so soft as to be readily excavated
and shovel. The amount of such disintegrated rock overlying the
varies greatly; in some cases the removal of a few inches will disclose
rock, in other cases the layer of decayed rock may be many feet in thick-
st-borings in rotten rock give samples similar to the samples from solid
that it frequently happens that while the foundations are planned for
the excavations disclose a thick layer of rotten rock. In such cases,
practicable to carry the footings down to solid rock, it may be necessary
e the size of the footings or to adopt some other expedient.

Rock. Where a rock-mass detached from the ledge of which it orig-
med a part is encountered it must not be loaded in excess of the safe
of the material by which it is surrounded. If the voids between ad-
ieces of loose rock are completely filled in with hard-pan, compact
nd, or clay, the loading may be the same as for the filling-in material,

rock, they will safely sustain any load which will not crush them.

Gravel. The name GRAVEL is given to rock-particles larger than smaller than the rock-masses known as BOULDERS. If compact, underlying bed of poorer material exists, gravel forms a most desirabl tion-bed, equal to sand or boulders in supporting power and not as be disturbed by adjoining excavations or pumping operations. If it may partake of the quality of hard-pan or rock. Care, however, taken to determine whether or not the bed of gravel has been deposit layer of silt or quicksand. It is possible for this dangerous conditio (See page 134.)

Sand. Sand is composed of comminuted rock-material. As qu most abundant rock-mineral and as its hardness and insolubility mak resistant to disintegrating action, it will be found to be the principal of most deposits of sand or sandy material. Grains of mica, felds and other minerals are frequently found. Sand is described as b MEDIUM, or COARSE, according to the size of the grains of which it is

Coarse Sand may contain particles of gravel, but after elimi particles which will not pass a screen with 4 meshes to the inch it wi that a large proportion of the remaining material is too coarse to pass sieve.

Fine Sand, on the other hand, may contain no particles which wi a 20-mesh sieve, and a considerable proportion which will pass a 100-

Very Fine Sand is frequently mistaken for clay and, indeed, gen contain some clay, as clay generally contains fine sand.

Uniform Sand is sand in which there is relatively a small variat size of the particles.

Balanced Sand is sand in which the size of the particles varies fro small and in which there is no great difference in the numbers of p each size.

Clean Sand contains no clay or loam, but a pure sand containing a centage of fine particles is often considered to be NOT CLEAN.

Sharp Sand is clean sand containing coarse, angular grains. Wh

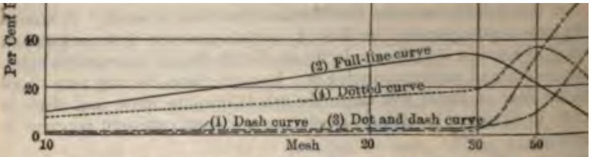

Fig. 1. Graphical Illustration of Results of Sieve-tests on Sands

uniformity of the size of its grains. For purposes of comparison an
to show the variation in sands which appear to be substantially the
dotted curve (4) has been added. This shows the result of tests on a
apparently as coarse as sand (2), but containing a much larger per
fine particles between 0.015 and 0.0055 in in diameter. Fine sand
contains a considerable proportion of clay. A chemical analysis of
QUICKSAND from the down-town section of New York City, reported
writer by Dr. C. F. McKenna, is as follows:

Mark: "Commercial Cable"

Silica. . . . .
Alumina and oxide of iron. . . . .
Lime. . . . .
Magnesia. . . . .
Loss on ignition. . . . .

A rational analysis shows the following composition:

Quartz, as given. . . . .
Clay and mica, as given. . . . .
Feldspathic detritus. . . . .

On the other hand, a sample of extremely fine sand from Michig
75% passed a 200-mesh sieve, appears to be absolutely pure quartz

Clay. When pure, clay consists of hydrated silica of alumina,
impurities are m

into the voids in the rock-fill, and this action will be continuous unt
the voids are filled in. Any vegetable matter, or other matter liable t
and shrinkage in volume, will increase the total shrinkage of the mas
tain natural deposits, such as beds of peat or soils containing vegetable
are apt to shrink in volume from the same causes. When it is nece
found a building on such material it is inevitable that the footings wi
with the mass, notwithstanding that the unit load on the foundation-b
small as to be negligible. In such cases the settlements may be vert
uniform; but if the depth of the fill under one part of the building is
than the depth under another part, the settlements will not be uniform
shrinkage in the fill will, in general, be in proportion to the depth of
No important building should be founded on such material and,
possible, the footings should be carried down through the filled-in ma
some more reliable underlying stratum.

8. Allowable Loads on Materials of Foundation-Beds

General Considerations. Owing to the infinite number of variatio
materials encountered and the conditions affecting the reliability of su
rials, no general or definite rule can be given, and every case should be
investigated before determining the allowable unit load on the foundat
If the material and conditions are uniform over the entire site of the
a uniform unit load may be used, but in practice it is frequently fou
entirely different conditions exist under different portions of the same
and in such cases great care must be exercised in determining the uni
For instance, one section of a building may rest on rock and another se
a light compressible soil or on a clay of doubtful stability. In such c
unit load on the compressible soil or on the clay must be reduced as
possible so as to reduce the differences in settlements between the two
of the building to a minimum. If the entire building were on a comp
soil a very considerable settlement might be allowable, provided it was
but in this particular case it is known beforehand that the part of the
on rock will not settle at all and that any settlements of other parts

ed building and of the material composing it, should be borne in ecting the unit load for any given foundation-bed, irrespective of the ssure as given by building codes or by examples quoted in this chapter.

ads on Rock. The safe unit load on rock may often amount to the crushing strength of brickwork or stone masonry, and in nearly y material worthy of the name of rock is capable of supporting from ns per sq ft.

ads on Sand, Gravel and Boulders. When compact and con- lly these materials are capable of supporting 10 tons per sq ft with- able settlement. It rarely happens, however, that it is advisable to aterials with more than 5 tons per square foot.

ads on Loose Sand. By loose sand is meant sand which has oroughly compacted and which may settle by its own weight inde- of a superimposed load. All such materials should be tested and d reduced in accordance with the result of such tests.

n Fine Sand or Quicksand. It is probable that fine sand, if confined, will sustain as heavy a load as coarse sand, but in view of at if afforded the slightest opportunity it is liable to lateral displace- inadvisable to found any structure on such material. When it is to place the footings on such material the unit load should be reduced possible and preferably to less than 2 tons per sq ft, and great care taken to connect all footings with a continuous layer of concrete so ent any flow of material into the cellar-excavation. Care should lso, that any sumps, pump-pits, drainage-arrangements and sewer- s for the building do not permit the escape of any quicksand.

ads on Hard-pan and certain cemented sands partaking of the hard-pan may approximate rock in hardness and reliability. Such however, are liable to soften if exposed to water. If these materials, vered, are dry, experiments should be made to determine how they en wet, and if the level of the water in the ground is liable to change ch the layer of hard-pan, the load should be correspondingly reduced. hard-pan containing gravel has been frequently loaded with more ns per sq ft. Care should be taken, however, to determine that hard-pan is continuous to a solid substratum, as it frequently happens of hard-pan and fine sand or clay are deposited alternately.

ads on Clay. Ordinary clay should not be loaded with more than sq ft. If soft and plastic, a load of 2 tons per sq ft may produce e settlements. Clay containing so large a percentage of sand as to sticity has been loaded with from 4 to 6 tons per sq ft without undue , and sand or gravel containing sufficient clay to act as a cementing ill partake of the qualities of hard-pan. In general, however, clay dangerous of all the materials on which structures are founded and ad should be reduced to a minimum and every precaution taken to flow of material. Undue reliance should not be placed upon load- clayey soils. It is probable that a loading on a large area which e a movement of the clay will on a small area have no effect, so that to rely upon the results of a test-load applied to an area smaller than supporting areas to be used. From the experience gained in the n of large buildings in Chicago which were FLOATED on clay, the unit load has been generally reduced to 2 tons per sq ft and, in s experience, a load of less than 2 tons per sq ft on clay has pro- ements varying from nothing to 12 in.

Special Requirements of Some Building Codes. The Boston c
vides that "the footing shall not overload the material on which it rest

The New Orleans code limits the maximum load to 1 400 lb per sq ft, t
city being on an alluvial-delta formation.

The Buffalo code limits the load on SOIL to 3½ tons per sq ft; if the soi
than hard clay or gravel the supporting areas "shall be extended as d

The Cincinnati code limits the load on soils INFERIOR to those listed,
per sq ft.

10. Investigation of the Site

General Considerations. To determine the character of the materi
will be encountered at the level of a foundation-bed, the architect sh
get as definite information as possible from others as to their experience
ing excavations and erecting buildings in that vicinity. In some local
subsoil conditions are uniform over large areas, while in other localitie
tant variations may occur within the limits of a city lot. Abrupt ch
surface-topography, changes in the character of the surface-soil or in t
vegetation, proximity to old or existing water-courses are suggestive
surface irregularities. In such cases, and in all cases where there is a
as to subsurface conditions, a sufficient number of exploratory boring
pits should be made to determine the facts. This exploratory work s
below the level of the proposed footings, should determine the grou
level and insure that no unsuspected layer of quicksand or other u
material underlies the foundation-bed. The methods in use for such
tions are as follows:

Testing in an Open Pit. For shallow work an open pit is t
satisfactory method as it allows actual inspection of the undisturbed
over a considerable area. If the excavation is in firm material, no sh
or other protection may be required; but if in flowing material, or i
deeper than adjoining footings, timber sheeting or steel sheeting s
employed. If the excavation is carried no deeper than the proposed
level, the underlying material should be tested by one of the method
after described.

driven so as to keep close to the bottom of the hole made by the auger.
pipe prevents the material falling from the sides of the hole and the au
cavates and loosens the material ahead of the pipe and facilitates driving.
above methods are not generally successful for deep holes or where
boulders or compact material interferes with driving the pipe.

Testing with Wash-Pipes. For test-borings over 10 ft in depth the
in most frequent use is the WASH-PIPE METHOD. In this method a wroug
or steel pipe known as the CASING-PIPE or DRIVE-PIPE is driven into the
much the same way as in the DRY-PIPE METHOD, but the driving of the
facilitated by the use of a JET OF WATER. The lower end of the casing-
provided with a hollow SHOE or reinforcement, slightly larger in outside di
than the casing. This serves to protect the pipe from injury in driving t
gravel or hard-pan, and forms a hole slightly larger than the diameter
casing. The upper end of the drive-pipe is protected from injury by an a
drive-head which has a threaded part fitting the thread on the casing-pi
a central hole to admit the jet-pipe. The jet-pipe is small enough to
it to freely enter the casing-pipe. The lower end is contracted so as to p
a jet-action. The upper end is connected with a water-supply which m
under considerable pressure. The driving-mechanism consists of a ca
weight with a central vertical hole large enough to admit the wash-pip
stationary verticals supporting a BLOCK-AND-FALL and an arrangement
releases the weight when it has reached a predetermined height. Wi
arrangement, water is continuously pumped through the jet-pipe, the le
which is regulated so that the jet-action loosens the material immediately
or AHEAD of the casing. Some of the jetting water returns to the surfa
side of the casing and thus lubricates the surface in contact with the
material. Another part of the water returns to the surface in the a
space between the wash-pipe and the casing, carrying with it particles
material loosened by the jet. As the jet loosens and washes away the m
immediately below the casing, the latter is driven deeper by repeated bl
the ram, the driving and washing being carried on at the same time.
operation is thus continuous until the top of the casing comes close to t
face of the ground, when the hammer drive-head and hose-connection

or sand in bags, or any other convenient material. The guys should than four in number, should be attached to the top of the post and horizontally so as not to pull up or down on it. Levels should be point on the post above or below the load, as may be most conven load should be applied gradually and with the least possible jar, care b also, to keep the loading uniform on opposite sides of the post, whic always vertical. Levels should be taken at frequent intervals during t tion of the load. The level observed when the platform is first in p be taken as zero and successive settlements referred to it. When th unit load has been reached, no additional load should be added unti settlement is observed. After this, first 50 and later 100% overl added and the total and periodic settlements observed. If the settle a test-load of twice the proposed load is not excessive, the test is cons factory.

12. Topographical and Special Conditions

Excavations over Inclined Strata. In case the site of a propo is on a slope, and especially if the slope is steep, there may be da slip of the material forming the foundation-bed. (See, also, page may occur if there is an inclined plane of separation between layers o lying rock, or between the rock-surface and the material overlyin or if inclined strata or beds of clay occur below the foundation-be such locations are the more likely to occur if water is present, as increases the weight of the soil and also reduces the COEFFICIENT o against sliding. Such conditions are frequently indicated by the ap springs or springy ground below the site. Where the base of the sl a stream or river there may be danger from the washing away of b have been supporting the side slopes of the valley. In the case of d with steep clay banks, or in any location where landslides have bee occur, great care should be taken to extend the footings to a bed th be affected by any landslide. It sometimes happens that there is a tinuous and general movement of the material forming the side slope toward the center of the valley; but such conditions are rare, fortun of real estate and damage to high priced properties. The depth of will generally be as shallow as practicable. Where the tunnel has to underneath some obstruction, the approach-grades will probably be imum or limiting grade of the particular section.

Relation of Subways to Foundations of the Most Importa ings. In SUBWAY-CONSTRUCTION for rapid-transit passenger servic can be operated on sharper curves and with steeper grades than wo in the case of a trunk-line railroad. This permits the lines to fol the lines of the city streets. For traffic-considerations the locati general, follow the principal arteries of surface traffic, and stations eral, be located at intersections of important streets, where there is congestion of population. As such conditions are caused by the trade-centers, and call for the construction of high buildings, it may seen that the heaviest and most important buildings are most lik their foundations affected by the construction of a subway in their vicinity. Where there is reason to apprehend the construction of WAYS OF TUNNELS, information should be sought as to the probab the excavation, the depth at which water is encountered, the chara material, the probable width of the construction as affecting the use vaults, and the method to be employed in making excavations. excavations for such tunnels and subways have been carried below of the footings of adjoining buildings, as in Baltimore, Boston, Brookly and New York City, buildings along the routes have been serious Such results have not been limited to any particular methods used struction of the tunnels, as even where the excavations were wholly in rock, serious damage has been done.

13. Loads Coming on the Footings

The Loads to be Considered in the design of the footings of a are:

(1) **The Dead Loads,** or the loads due to the actual weight of the structure, ready for occupancy.

(2) **The Live Loads,** or the loads due to the occupancy of the buildin

(3) **The Wind-Load** is generally calculated on the assumption tha
may exert a uniform pressure, frequently taken at 30 lb per sq ft, on
external area of any side of the building. This assumption makes no
for the protecting influences of adjoining buildings. In a building of
is improbable that the maximum pressure will be reached over the enti
area at the same instant of time, and consequently, if the assume
represents the maximum pressure, the average, at any time, will be
the calculated total.

General Effect of Wind-Pressure. The horizontal pressure of the
to increase the load on footings on the leaward side of the building
crease the load on footings on the windward side. In many building
bracing, called wind-bracing, or other special construction, is used
the building from being deformed by the wind-pressure and to conver
zontal stresses due to the wind-pressure into vertical components, ac
defined lines of support, that is, into either uplifts or loads on cer
piers or columns. Where the uplift on any element of the structure i
the dead load on the same element, the uplift is ignored. Where t
component increases the compression in any element it is called the WIN
THAT ELEMENT of construction and on the corresponding footing. T
is generally based upon concentrating all of the wind-load on certai
footings. If, on account of the general rigidity of the building, or on
any other reason, the wind-stresses reach footings not designed to rec
loads, the amounts figured on the external footings will be reduced co
ingly. It is probable that the maximum effect of the wind results fro
of impulses of short duration and that the effect of such pulsation
partially overcome by the inertia and elasticity of the buildings; so
resultant load reaching the footing may be only a part of the theoretica
the instant during which the maximum pressure is exerted. (See, also,
XXIX, Wind-Bracing of Tall Buildings.)

The Probable Maximum Wind-Load acting on the footing is, there
than the theoretical load due to the maximum wind-pressure. If the
wind-load represents approximately the maximum wind-pressure, as
by a wind-gauge, it would appear safe to assume that only 50% of the
wind-load would act to produce a settlement in the footings of a

14. Assumed Loads Specified by Building Codes

II. Requirements of Building Codes for Assumed Loads for Office-Buildings

City	Requirements
ta, Ga.	Live load, 75 lb per sq ft above 1st floor; 150 lb per sq ft on 1st floor Footings designed for dead load and 60% of live load and wind-load
n, Mass.	Live-load, 100 lb per sq ft. Wind-load, 30 lb per sq ft where erected in open spaces; in built-up districts, 25 lb at the 10th story, 2½ lb more for each succeeding upper story, up to a maximum of 35 lb to the 14th story and above
o, N. Y.	Live load, 70 lb per sq ft. Wind-load, 30 lb per sq ft. Foundations designed for the acting average loads in the completed and occupied building and not the theoretical or occasional loads
apolis, Minn.	Live load, 75 lb per sq ft above the first floor; 100 lb for first floor. Wind-load, 30 lb per sq ft. Roof and top floor, full live load. For each succeeding lower floor, a reduction of 5% until 50% is reached, such reduction being used for the remaining floors
nond, Va.	Foundations designed for 60% of the live load
uis, Mo.	Live load, 70 lb per sq ft; first floor, 150 lb. Loads carried by the soil, total dead load and 10 lb per sq ft of all the floor-area. Wind-load, 30 lb per sq ft
ul, Minn.	Live load, 60 lb per sq ft above the first floor. First floor, 125 lb. Wind-load, 30 lb. Roof and top floor, full load; for each lower floor, a reduction of 5% until 50% of the full live load is reached, when such reduced load shall be used for the remaining floors. Footings designed for dead load and live load
nnati, O.	Live load, 50 lb per sq ft above first floor; 100 lb for first floor. Live load reduced by 5% for each floor below the top until 20% is reached, when such reduced loads shall be used for remaining floors. Wind-load, 20 lb per sq ft above surrounding buildings
go, Ill.	Live load, 50 lb per sq ft. 50% of the live load used for piles. Piers designed for 85% of live load on top floor and reduced by 5% for each lower floor until 50% is reached, when such reduced loads shall be used for the remaining floors. Wind-load 20 lb per sq ft
York City	Footings designed for 60% of the live load
eland, O.	Live load 60 lb per sq ft in offices proper. 100 lb per sq ft in halls, lobbies, etc. Footings for walls designed for 50% of live load. Free-standing columns designed for 80% of 100-lb load and 75% of 60-lb load. Wind-load 30 lb per sq ft for free-standing structures in built-up districts; 25 lb per sq ft at the 10th story and 2½ lb less for each lower story, and 2½ lb more for each higher story, until 35 lb is reached

The Minimum Areas of Support. The actual dead loads and the a live loads and wind-loads for each linear foot of wall and for each colum or other supporting element of the building down to the level of the having been calculated, a foundation-plan should be prepared giving the and center of action of all loads. For safety under the worst possible co tion of loads, each footing should be ample to support the total of the dea live loads and wind-loads coming on it. The MINIMUM AREAS OF SUPP any footing are obtained by dividing the total of the dead loads, live loa wind-loads by the safe supporting power of the foundation-bed. foundation-bed is rock, or can be considered as incompressible under t load, the minimum areas so obtained may be used for the footings. O pressible materials, or generally on all materials other than rock, the these minimum areas will not result in uniform settlements owing to t that the actual live loads and wind-loads are not consistent with the a live loads and wind-loads.

The Actual Loads on the Footings. In accordance with what h previously said, let us assume that the dead load is constant, and tha building under consideration the probable maximum live load is 50% assumed live load, that the probable maximum wind-load is 40% of the a wind-load, and that on the completion of the building, for a short peri live loads and wind-loads reduce to zero. The ACTUAL LOADS ON THE FO would then be:

(1) Upon completion of the building, the dead load only;

(2) Under the maximum load due to occupancy and to snow on th the dead load plus 50% of the assumed live load;

(3) When loaded as in (2) and subject, in addition, to the maximum able wind-action,

  (*a*) The footings on the leaward side of the building will sustain th dead load, plus 50% of the assumed live load, plus 40% of sumed wind-load;

  (*b*) The footings on the windward side of a building will sustain th dead load, plus 50% of the assumed live load, minus 40% of sumed uplift;

  (*c*) Other footings will support the total dead load, plus 50% of ting areas having a lower percentage of live loads and wind-loads will higher unit load, and the corresponding footings will have settled more ther footings supporting a high percentage of live loads and wind-loads. s basis, the footings will not come to a uniform settlement until they een subjected to the maximum live loads and wind-loads.

itrary Rules for Proportioning Supporting Areas. Various ARBI- RULES have been recommended for the proportioning of the supporting o secure equal settlements. These rules generally provide for a reduc- the assumed live loads and wind-loads, but do not take into consideration t that a large proportion of the total settlement of certain footings may lace subsequently to the completion of the building and after other foot- ay have reached practically their full settlement.

onal Rule for Proportioning Supporting Areas. The rule herein- ecommended provides not only for a reduction of the assumed loads on rational basis, but also for the proportioning of the footings for the mean nstead of for the ultimate load, and it is believed that the resulting settle- will be as nearly uniform as possible. The rule is based on the propor- of the footings in accordance with the loads which will act on the s at the time when all of the dead loads and one-half of the probable um live loads and wind-loads exist. The reason for taking one-half of obable maximum wind-loads and live loads is that these loads vary from a maximum, the average being one-half of the maximum.

vision for Variations in Loads. On the completion of the building fore the live loads or wind-loads have gone on the footings, the settle- will not be uniform, because areas designed for a high percentage of live nd wind-loads will have much less than their average load and will there- ve settled less than footings having a low percentage of live loads and wind- When these same footings have been subjected to the maximum probable ads and wind-loads, the settlements will again be unequal, because the have been proportioned for only one-half of the probable maximum live and wind-loads; but the footings which originally were the highest will e the lowest. The inevitable movement due to the variation in the live and wind-loads will be equally divided, one-half of the settlement being d to bring the footing to the level of a footing having the dead loads only,

areas rather than by diminishing any. Any mean unit load which wo
when divided into the mean loads, areas, all of which would be larger t
minimum areas, would serve as the mean unit load, but it is more eco
to determine the LOWEST POSSIBLE MEAN UNIT LOAD which, when ap
the mean loads, will give the least possible increase of the areas. Thi
done by determining which one of the minimum areas carries the LEA
LOAD PER SQUARE FOOT. This area may be selected by calculating t
load on each of the minimum areas, or more simply, by comparing t
of assumed loads and a table giving the mean loads, and noting which
has the LARGEST PERCENTAGE OF REDUCTION between the assumed load
mean load. The resulting mean load on this footing will be the MINIM
LOAD which can be used as a MEAN UNIT LOAD.

The Method Reduced to Rule. The method can be reduced to
follows:

(1) Prepare a table giving in vertical columns or table-divisions
footing, the dead loads, the assumed live loads, the assumed wind-lo
the total of these three loads. This table is called the TABLE OF ASSUME

(2) Prepare a similar table giving the dead loads, one-half of the m
probable live loads, one-half the maximum probable wind-loads and
of these three loads. This table will be called the TABLE OF MEAN LOA

(3) By a comparison of the two tables, find the supporting area w
suffered the greatest percentage of reduction between the total assum
and the total mean loads and find the unit load resulting from the mean
this area. This unit load will be called the MEAN UNIT LOAD.

(4) Divide the total mean load as given in the table of mean loads
footing by the mean unit load. The result will be the required AREA
PORT.

Short Method for Determining the Mean Unit Load. From t
going it follows that the MEAN UNIT LOAD can be obtained more directly by
lowing rule. Find the supporting area which has suffered the largest per
of reduction between the total assumed load and the total mean load an
ply the allowable unit load on the foundation-bed by the ratio obta
dividing the total mean load by the total assumed load.

= 25% of the assumed wind-loads. Table IV gives the dead loads a
mean live loads and wind-loads separately, and the total of the dead lo
mean loads, which total is to be used in proportioning the areas for leas
tion in settlement. This is known as the total mean load.

Table IV. Dead Loads, Mean Live and Wind-Loads and Total Dea
Mean Loads

Element of footing	Division 5, dead loads, unchanged, lb	Division 6, one-half of 60% of assumed live loads, lb	Division 7, one-half of 50% of assumed wind-loads, lb	Divi total loa
Side walls per lin ft....	14 000	2 520	500	1
Columns 1 and 5......	137 500	48 000	....	18
Columns 2, 3 and 4....	90 000	102 000	....	19
Chimney..............	320 000	....	....	3

Table-columns are called divisions to avoid confusion with building-columns.

Comparing the two tables it will be seen that the interior column
building, columns 2, 3 and 4, had originally the largest percentage of li
plus wind-loads, and have consequently suffered the greatest reductio
amount of total load. The minimum areas of support for columns 2,
and also for the other elements of the footings, are obtained by divid
total assumed loads given in division 4, Table III, by 8 000, the a
unit load in pounds on the foundation-bed. The resulting areas are
division 9, Table V. No reduction can be made in these areas without
ing the limitation that the most disadvantageous combinations of loadin
ever improbable, shall not exceed the safe unit load. The adjustment
areas to the probable mean loading, as given in Table IV, the table o
loads must be accomplished solely by increasing the sizes of certain foo

e Table IV, the table of mean loads, with Table III, the table of assumed ad find the element of support which has suffered the highest percentage tion between the total assumed load and the total mean load, and note esponding minimum area of support at the allowable unit load on the ion-bed. Divide the mean load for the same element of support by ber of square feet in the minimum area of support. The result will be load for mean settlement. Then divide the mean loads for each element of support by the mean unit load. The results will be the required areas in Table V.

e V. Mean Loads on Minimum Areas and Areas for Mean Loads

lement of footing	Division 9, minimum areas, sq ft	Division 10, mean loads on minimum areas, lb per sq ft	Division 11, areas for mean loads, sq ft
alls per lin ft	3.05	5 580	4.7
ns 1 and 5	37.2	4 986	51.9
ns 2, 3 and 4	53.8	3 568	53.8
ey	40.0	8 000	89.7

columns are called divisions to avoid confusion with building-columns.

e mean unit load may be determined by multiplying the allowable unit the ratio obtained by dividing the mean load for the element of support ving suffered the highest percentage of reduction by the assumed load same element.

lting Settlements. The following Tables VI, VII and VIII show the ative settlements which may be expected if the supporting areas are ioned in accordance with different assumptions as to load. In all the is assumed that the foundation-bed will settle ½ in per ton of load, and total assumed load will never load the foundation-bed in excess of 4 tons t.

ble VI the footings are proportioned in the ratio of the DEAD LOADS only.
ble VII the footings are proportioned in the ratio of the TOTAL ASSUMED

ble VIII the footings are proportioned in the ratio of the MEAN LOADS.
ch table, division 1 gives the dead load coming on the footings on the ion of the building. Division 2 gives the load coming on the footings e building is subjected to the maximum probable live loads and wind- Division 3 gives the supporting areas in accordance with the assumed . Division 4 gives the settlements for the unloaded building. Divi- ives the settlement after the addition of the maximum probable live d wind-loads.

nation of Table VI. The method of proportioning the areas in the dead loads only, as recommended by C. C. Schneider* may, in the form e, be stated as follows:

Then, 90 000 ÷ 53.8 = 1 675 lb, the unit load due to the dead load o
area for columns 1 and 5 is 137 500 ÷ 1 675 = 82.1 sq ft. The proce
lar for the other elements.

Table VI. Footings Proportioned in the Ratio of the Dead Loads

Probable settlement where supporting areas are proportioned in the r
dead loads only

Element of footing	Division 1	Division 2	Division 3	Division 4
	Dead loads only, lb	Maximum probable loads, lb	Areas, sq ft	Settle
				Empty, in
Side walls per lin ft.....	14 000	20 040	8.3	0.42
Columns 1 and 5........	137 500	233 500	82.1	0.42
Columns 2, 3 and 4.....	90 000	294 000	53.8	0.42
Chimney...............	320 000	320 000	191.0	0.42
Maximum variation, empty....................			....	0.00
Maximum variation, loaded....................			....	....

Table-columns are called divisions to avoid confusion with building-colum

The calculations for settlements are readily made, when the amoun
pressibility of the foundation-bed is known, by multiplying the unit lo
foundation-bed of each element of support by the amount of compres
the foundation-bed per unit of load. Thus, in the above example th
of compressibility is given as ½ in per ton. In Table VI the unit loa
dead loads for each element of support, are the same, or 1 675 lb = o

General Requirements. In laying out the AREAS OF SUPPORT for
ture it should be borne in mind, as previously explained, that (1) th
the dead loads, assumed live loads and assumed wind-loads should n
foundation-bed in excess of the allowable load on it; (2) when the f
bed is compressible the areas of support should be calculated by the
mean loads; and (3) the center of gravity of the supporting area shou
with the center of action of the load to be supported. To these ma
a further condition that (4) economy will be furthered by keeping th
ing areas simple in outline and by arranging each area as compactly
around the center of the load to be supported.

(1) The first condition is necessary in order to provide that no p
dition of loading will exceed the allowable pressure on the foundatio

(2) The second condition provides for making the settlements c
footings as nearly equal as possible.

(3) The third condition provides that the settlements of each fo
be uniform, that is, that the footing shall not settle out of level.

(4) The fourth condition provides for economy in design in the fo
and for economy in making the excavation for the footing, especially
of deep excavations requiring sheathing for the protection of their si

In the case of a free-standing structure, the total load of which is n
of the supporting capacity of the entire area of the building at th
load on the foundation-bed, it will generally be possible to arrange s
porting areas whose centers will correspond with the centers of the l
disposition of such areas is considered in succeeding paragraphs in
sions of CONCENTRIC LOADING. In buildings having restricted si
walls or columns are placed close to adjoining property-lines, it will
be impossible to arrange for simple concentric loadings and necess
offset footings, cantilevers or other devices to transfer the loads to
areas located on the property. Such supporting areas are discussed i
ing paragraphs relating to ECCENTRIC FOOTINGS.

Footings with a Concentric Load. In order to have the lo
foundation-bed uniform under a footing it is necessary that the cente
ity of the supporting area should coincide with the center of gravity o
otherwise the area is said to be ECCENTRICALLY LOADED and the resu

ling movements in the grillage and s reduced to a minimum.

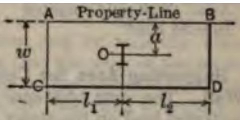

Fig. 3. Elongated Supporting Area. Concentric Load

ated Supporting Areas. When the ig area for an isolated load cannot be circle or a square, for example, when re or the circle would overlap an ad- property-line or interfere with an ad- supporting area, the necessary area quently be made RECTANGULAR in form, as *ABDC* (Fig. 3), having a , twice the distance *a* between the center of the load *O* and the ine *AB*.

quired length *l* equals the required area divided by *w* and the area centered on *O*, that is, l_1 must equal l_2.

inations of Simple Areas. Two Adjacent Isolated Areas. When supporting areas overlap or when, for other reasons, it is desirable to ADJACENT FOOTINGS, the best arrangement may be obtained as follows: the supporting area required for each of two adjacent concentrated d the distance between the centers of the loads, the sum of the two uld be divided by twice the distance between the load-centers. The will be the width or the dimension of the required rectangle of sup- en at right-angles to the line connecting the load-centers; and the nension of the rectangle will be twice the distance between the load- The center of the area should be placed so as to coincide with the gravity of the two loads, when it will be found that each load will ntric with its own area of support. Where a row of columns requires ich nearly overlap, the COMBINATION OF THE AREAS frequently results in in excavation and form-work.

rting Area for a Concentrated Load in the Line of a Wall. more concentrated loads are carried in the line of a wall the ADDITIONAL NG AREAS required for such concentrated load may be provided in two ways.

the concentrated loads rest on the wall, as, for example, when the wall the ends of girders and when the conditions are such that the con- l loads are distributed along given lengths of it, then, all that is neces-i: is desirable that the additional areas, together with the area for th between them, should APPROXIMATE A SQUARE. Knowing the wi footing required to support the wall and the additional area required the concentrated load, the length of the required square can be determined by th formula (Fig. 4):

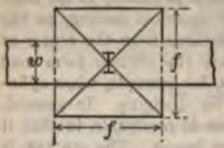

Fig. 4. Square Supporting Area. Wall and Concentric Isolated Load

Let w = the width of the footing;
A = the area required to support trated load;
f = the side of the square which a length of wall equal to provide an additional area
Then

$$f = \tfrac{1}{2} w + \sqrt{A + \tfrac{1}{4}}$$

Supporting Area for Concentrated Load not in the Center Wall. The same additional supporting area is required for this as centrated load on the center line of a wall, but the total area must unequally between the two sides of the wall-footing, the larger portion being placed on the side of the eccentric load. The simplest way to determine the location of the supporting areas for this combination is to determine the size of the required square as if the concentrated load were concentric with the center line of the wall. The next step is to calculate the load due to the wall for the length of this square and determine the location of the center of gravity of the combined loads, that is, the center of gravity of this wall-load and the concentrated load. The center of the supporting placed concentrically with the center of gravity of the combined Fig. 5 let

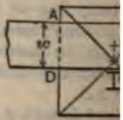

Fig. 5. Square Area. Wall tric Isolated

w = the required width of the wall-footing;
O = the concentrated load;
A = the area required for the support of the concentrated lo as before, the length of the side of the required square

$$f = AB = \tfrac{1}{2} w + \sqrt{A + \tfrac{1}{4} w^2}$$

The center of gravity of the wall-load contained between the lin BC is at g, and the amount of the load is evidently the load per foot by the distance $AB = f$. Knowing the po amount of the loads at O and g, the center of the combined loads is determined, say a fixes the center for the square.

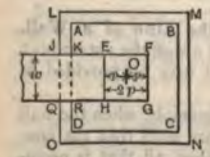

Fig. 6. Square Supporting Area. Isolated Load on End of Wall

Supporting Area for a Concentrated the End of a Wall. A somewhat different is required for this, but the supporting are best determined as follows (Fig. 6): Kn width w of the footing required for the supp wall, the additional area required for the co load O and the distance p from the center of t trated load from the end of the wall, proceed in this way. Determine whose area corresponds to the sum of the areas required for the the concentrated load and for a length of wall equal to twice the projec wall beyond the center of the concentrated load. Plot this square AB

be $K_1 = W/L\,(1 + 6\,U/L)$, or the MAXIMUM LOAD will equal the averag plus six times the average load multiplied by the ratio of the ECCENT divided by the width of the footing.

Similarly, the pressure at B, the edge farthest from the point of appli of the load, will be $K_2 = W/L\,(1 - 6\,U/L)$, or the MINIMUM LOAD equa average load minus six times the average load multiplied by the ratio eccentricity divided by the width of the footing.

When the ECCENTRICITY equals 1/6 the width, the pressure at B become If the eccentricity exceeds 1/6 the width there will be an uplift at B, or the ing will have a tendency to overturn. This relation is generally expres saying that to avoid an upward reaction the line of the load must fall within the MIDDLE of the base.

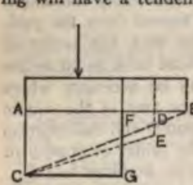

Fig. 8. Pressure-diagrams for Footings

Load-Diagrams for Offset Footings. If i diagram (Fig. 8) the figure *ADEC* represent load-diagram on the foundation-bed for a wi footing *AD* and the load *AC* is the maximum missible load, then the area *ADEC* represent maximum support afforded by the footing *A* the width is increased until the load falls on th of the middle third or to the width *AB*, the load at B is zero and the support is represented by the triangle *AB* area of which is less than the area *ADEC*. Moreover, if the wid the footing is reduced until its center is concentric with the load-center the load-diagram becomes *AFGC*, the area of which is greater than either or *ADEC*. From the foregoing it is evident that any advantage gain offsetting the footing must be obtained at the cost of concentrating the s given to the wall away from the center line of the wall.

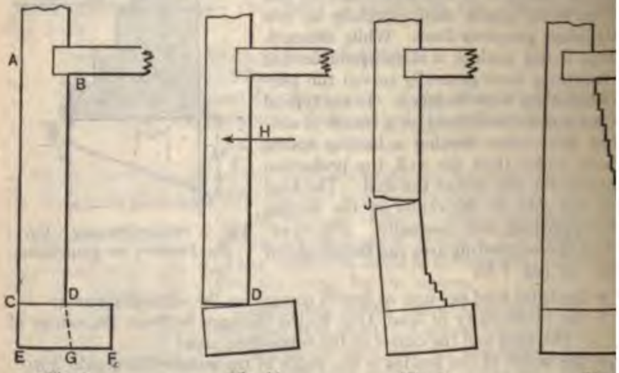

Fig. 9 Fig. 10 Fig. 11 Fig.

Figs. 9, 10 and 11. Eccentric Loading and Tendencies to Failure Due to Off Footings; Fig. 12. Improved Type of Construction

Eccentric Loading Due to Offset Footings. In Fig. 9, represen simple case of ECCENTRIC LOADING due to OFFSET FOOTINGS, the load foundation-bed at E is perhaps twice the average load and at F abou Under these conditions the projecting portion of the footing may shear, a

Determination of the Area of Support. In determining the SUPPORT for *A*, having assumed one dimension of the supporting twice the distance *GP*, or say 5 ft, the other dimension will be 105¼ sq ft ½ in. If the length 21 ft ½ in, as determined, is found to be exces

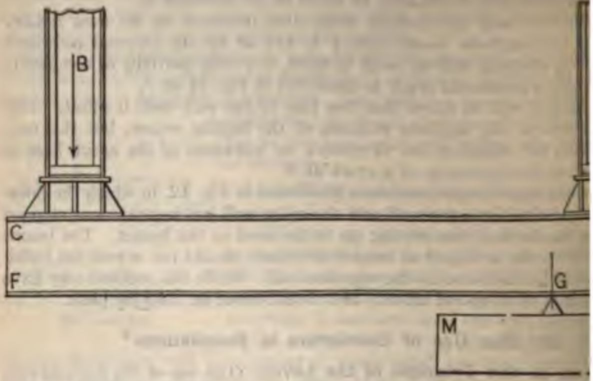

Fig. 13. Cantilever Foundation-construction

the point *G* must be moved to the left and the corresponding length porting area must be determined as before. When the length of the area for the fulcrum of the cantilever is limited, so that the length the property-line is fixed, the width of the area can be determined expe or by the use of the formula

$$X = (L - u) - \sqrt{(L + u)^2 - 2WL/lS}$$

in which

L = the distance between centers of the two loads;
W = the load nearest to the property-line;
l = the length of the supporting area;
S = the unit load on the supporting area; and
u = the distance between the center of action of the load to be ca and the edge of the supporting area nearest to the prop

If the position of the center of gravity of the load *A* combined part of the load on *B* which is borne by the cantilever is determined, found to coincide with the fulcrum or point of support *G* of the cantil demonstrating that the use of the cantilever provides a means of two loads so that their center of gravity falls on the center of a supp not concentric with either load.

The Grillage Fulcrum. Of course in practice the KNIFE-EDG shown in the diagram is not used. The bottom flange of the gir the cantilever rests on the DISTRIBUTING GRILLAGE directly, as is show which may be considered a typical arrangement.

The Girdering-Method for Two Equal Loads. W support two or more adjacent concentrated loads on a si the distance D between the columns plus twice the extension u. Knowing the length of the required area the width w is determined by simple division.

The Girdering-Method for Two Unequal Loads. In the case of columns not equally loaded, the SUPPORTING AREA may be a TRAPEZOID, as in Fig. the center of gravity of which must coincide with the center of gravity of the two loads. Knowing the sum of the two loads and the required area for the

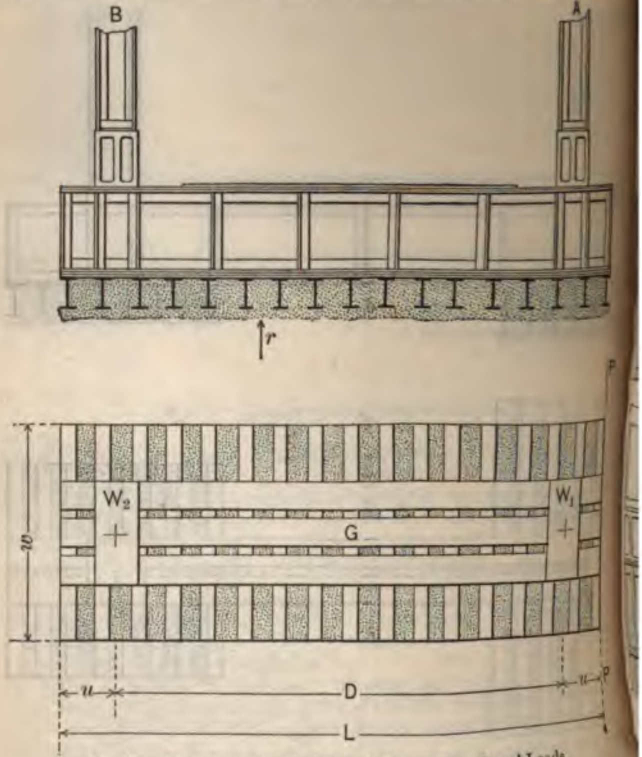

Fig. 15. Girdering-method of Foundations. Two Equal Loads

support, and fixing the total length L of the footing in accordance with requirements that the footing shall not project beyond the line PP, the wi of the footing at the small and large end, a and b respectively, can be deter as follows: Let B represent the distance from the small end of the tra to the center of gravity of the two loads and let A represent the area trapezoid. Then

$$b = 2A/L\,(3B/L - 1)$$

and

$$a = 2A/L\,(2 - 3B/L)$$

also

$$A = (a+b)\,L/2 \quad \text{and} \quad a+b = 2A/$$

load on the foundation-bed should be UNIFORMLY DISTRIBUTED and
the foundation-bed is compressible and the load concentric with the
area, it may safely be assumed as uniform, since a compressible ma
adjust itself until the loading at different points is substantially unif
unit load on the foundation-bed is evidently the total load divided b
porting area. If the area of the footing course varies between the to
tom of the footing the INTENSITY of the load will vary, and if uniformly d
the unit load at any level is obtained by dividing the total load by t
the footing at that level.

The Weight of the Footing Itself. This is generally so small
pared with the superimposed loads that it may be ignored without se

The Transmitting of Loads by Footings. If we neglect the
the footing we can consider the footing course as transmitting the im
to the foundation-bed or as being subject to two equal loads; one, t
IMPOSED LOAD, more or less concentrated on the center line of the fo
acting downward; the other, the REACTION due to the loading of the fo
bed uniformly, distributed over the supporting area and acting upwar
loads or forces being equal and opposite in direction, the stresses de
the footings are due to the differences in the distribution of these l
the footing courses simply act to convert concentrated into distribute

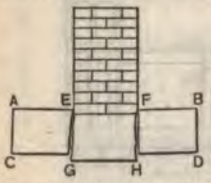

Fig. 17. Failure of Footing by Shearing

Manner of Failure of Footings. A foo
fail in several ways: (1) by SHEARING; (2)
CRUSHING; (3) by SPREADING; and (4) by
or RUPTURE.

(1) Failure of Footings by Shearing.
illustrated in Fig. 17, showing a wall the
which has caused it to SHEAR along the
and *FH*.

The force tending to cause SHEAR is the w
to the wall less the reaction of the foundatio
ing on the under side of the section *EFGH*.
load is supposed to be uniformly distributed this is equivalent to t
of the area corresponding to the width *CD* minus the width *GH* time
on-bed.

foundation-bed is of clay or other yielding material, which has, unde
of the footing, a tendency to FLOW along the lines indicated by arro
figure. This tendency should be provided against by making the bot
continuous and adequate to resist the tension. Vertical joints, su
made in footings composed of masonry, are sources of weakness, and
avoided. The TENDENCY TO SPREAD is greatest in footings having a spr
is wide compared with the width of the superimposed wall or other con
The writer knows of at least one important footing which has failed in
the cracks in general following the joints of the masonry substantially
in Fig. 18.

(4) **Failure of Footings by Bending or Rupture.** A footing m
BENDING or RUPTURE as a beam or girder. In the case of a wall, if
ing bends, as shown in Fig. 19, the concentration of the load on the lo

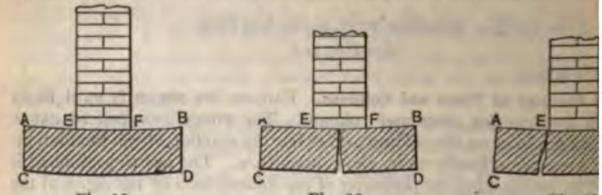

Fig. 19 Fig. 20 Fig. 21

Figs. 19, 20 and 21. Failures of Footings by Bending

of the wall, as at *E* and *F* may cause the base of the wall to fail. This p
should be borne in mind in designing footings where the load on the
proaches the allowable unit load for the material composing it, and
where the width of the footing is much greater than its own width.
footing fails by RUPTURE the rupture may occur either under the cent
the wall, as in Fig. 20, or at points close to the outer edge of the w
Fig. 21. Fig. 20 illustrates the objection to using a footing course c
of masonry or stones which do not extend the full width of the footi
joints in such construction prevent the footing course from acting in
and the footing as a whole from acting as a BEAM.

20. Methods of Calculating Bending-Stresses in Wall-Foot

Assumptions Made in Determining Bending-Stresses in F

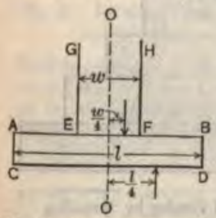

Fig. 22. Bending-stresses in Footings. First Method

Two methods for the calculation of the s
STRESSES IN FOOTING COURSES are in gene
Both are based upon the assumption t
REACTION of the foundation-bed is UNIFOR
the methods differ in the assumption mad
how the footing course and the base of the
structure act. Neither assumption can be
be wholly correct.

**The First Method of Determining B
Stresses in Footings.** This method is b
the assumption that the pressure of the w
footing is uniform over the area and remai
times.

If, in Fig. 22, *ABCD* represents a footing course supporting a cent
wall *EFGH*, and if

Then, by Equation (1), we have

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)\,11\tfrac{2}{3} = 50\,750 \text{ ft-lb}$$

If Equation (2) is used, we have

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)^2 = 42\,050 \text{ ft-lb}$$

and by Equation (3)

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)\,(11\tfrac{2}{3} - 1) = 46\,400 \text{ ft-lb}$$

Comparing the results we see that the moment by Equation (3) is
of the moments by Equations (1) and (2).

Graphical Comparison of Bending Moments in Footings.
a graphical comparison of the moments for varying ratios of l to
by Equations (1), (2) and (3) on the assumption that

w = the width of wall = 1 ft;
U = the unit load on the foundation-bed = 1 000 lb per sq ft
$r = l/w$.

The load on the wall, in pounds, for any value of l, is 1 000 l.

Comparing the curves of Equations (1) and (2) it will be seen that
are widely apart, the percentage of variation being highest in the
projections. When l is less than twice w, or in other words, when th
is less than one-half the width of the wall, Equation (2) gives momen
half the moments given by Equation (1). Equation (2) may be us
projections. Equation (1) gives results which are too large, espe
the projections are small. Equation (3), giving results half-way bet
of Equations (1) and (2) and in accordance with a reasonable hypot
appear to be preferable, but is not in accordance with present practi

21. Bending Moments in Footings of Columns and P

General Statement of the Problem. Fig. 25 represents in pl
column resting on a footing which projects on four sides. The
column or pier is represented by ABC
footing and its area of support by EF
part of the footing included in the areas
$QRST$ can be considered as acting in the
as projecting footings under a wall, but t
the four corners $EQMA$, etc., on which
posed wall-load is imposed, also caus
moments.

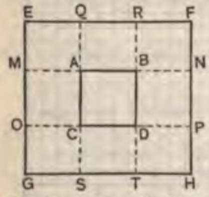

Fig. 25. Plan of Column-footing with Four Equal Projections

Different Theories. There are sever
more or less complicated and unsatisfac
how the UPLIFT ON THE FOUR CORNER-AR
be determined. The discussion of thes
would be out of place in this chapter.
footing, the projection is not over one-half the width of the sup
base, the four corner-areas will not aggregate over 25% of
area of the footing, and it may then be assumed that the bendin
is the same as if the base of the column or pier extended like a
the entire footing, as is shown in Fig. 26. To insure these conditions,
projection of the footing exceeds ½ w, and in all cases when the foot
homogeneous, as when a grillage of steel is used, the load of the co
be distributed over the width of the footing by a GIRDER or BOLSTE
extension of the column-base. In case the footing is in several

Compound Footings. In COMPOUND FOOTINGS where, for examp and a column or two or more columns are supported by a single f where loads are cantilevered, the loads will in general be distributed t porting area by GIRDERS or CANTILEVERS. The shears and bending of such girders or cantilevers must be determined for each case by th used in the calculations of beams and girders in Chapters XV and XX

22. Design of the Footings

Materials used for Footings. To possess the required strength MOMENT OF RESISTANCE of the footing must be at least equal to th OF RUPTURE, calculated as explained in the preceding paragraphs. whether of brickwork or stone, is not generally suitable for any but est buildings, as its tensional strength is low. Concrete, plain or or grillages of steel embedded in concrete, are generally employed. (ter III for footings for light buildings.)

Footings of Homogeneous Slabs. If the footing is composed OF HOMOGENEOUS MATERIAL, as a block of granite or other reliable buil or a single layer of concrete, the MOMENT OF RESISTANCE is, by the flexure-formula for rectangular cross-sections, $M_r = \frac{1}{6} bd^2 S$ (see Ch XV and XVI) in which

d = the depth or thickness of the footing, in inches;
b = the breadth of the footing, in inches;
S = the allowable unit tensile stress of the material, in pounds inch;
M_r = the moment of resistance.

Placing M, the moment of the forces tending to cause rupture, eq for a length of wall equal to 1 foot we have

$$b = 12 \text{ in}$$

and

$$d^2 = \tfrac{1}{2} M/S$$

Substituting in Equation (4) the value for M in inch-pounds as deter

$$l_1 = \frac{2\,M}{3\,W} + w = [(2 \times 15\,750)/(3 \times 28\,000)] + 2 = 2.375$$

Also

$$l_2 = 2.75 \text{ ft}, \quad l_3 = 3.125 \text{ ft}, \quad l_4 = 3.50 \text{ ft} \quad \text{and} \quad l_5 = 3.875$$

As l_5 is nearly 4 ft, the required length, it may be made so by in
thickness of the bottom course to 16 in. The total thickness of t
therefore (4 × 15 in) + 16 in = 76 in instead of 35 in, as previously
by Equation (1) for the footing cast as a unit.

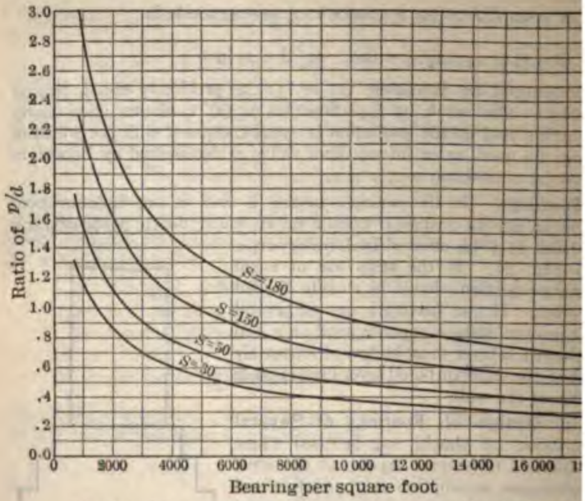

Fig. 28. Diagram Showing Ratio of Projection to Depth of Footi

l dimensions are in inches, S in pounds per square inch, and U in
square foot. The quantity p/d is the ratio of the projection to the
e beam or footing. For a given value of S the ratio will vary in-
he square root of U.

am (Fig. 28) shows curves for different values of S and U from which
projection to depth of footing may be taken. Thus, for a concrete
which the allowable unit stress, S, in tension is, say 30 lb per sq
ad, U, on the foundation-bed is 3 000 lb per sq ft, the allowable pro-
be 0.69 times the depth of the footing course. If the concrete is
the allowable offset will be 8.3 in. Conversely, for a given offset,
hen the unit load is 3 000 lb and $S = 30$ lb as before, the required
e 1.45 times the offset.

23. Steel Grillages in Foundations*

ges in the Use of Steel-Beam Grillages. When it is desirable
e deep excavation required for concrete or masonry footings, and
ad of a wall has to be distributed over a wide area of support, STEEL
EEL BEAMS are frequently advantageously used to give the required
resistance with a minimum of depth. Steel beams are generally
l preferable to rails, although second-hand rails have frequently been
expedient.

g the Bed and Setting the Beams. The foundation-bed should
ered with a layer of concrete not less than 6 in in thickness and so
compacted as to be as nearly impervious to moisture as possible.
should be placed on this layer, the upper surfaces brought to a line
wer flanges carefully grouted so as to secure an even bearing. Sub-
concrete should be placed between and around the beams so as to
y protect them.

nents for Steel Grillages. In determining the number and size
s for any given footing the following points should be considered:
beams must resist the MAXIMUM BENDING MOMENT, and this without

(5) Where the BENDING MOMENT is the governing feature, of two bea equal weight, the deeper beam should be used. Thus, if the required se MODULUS is 147, a 20-in 80-lb beam might be used; but a 24-in 80-lb bea stiffer and stronger in bending.

(6) Where the SHEAR is the governing feature, of two beams of equal we the smaller beam is the stronger. Thus, the SHEARING VALUE of a 20-in beam is greater than that of a 24-in 80-lb beam and is nearly equivalent to of a 24-in 90-lb beam. However, on account of the greater STIFFNESS of deeper beam it is sometimes advisable to use it even though the cost is incr

Spacing of Beams in Grillage. Table IX gives the LIMITING SPACING steel beams, based upon the safe capacity of the concrete filling acting a beam, for loads of from 1 to 6 tons per sq ft. Since, however, in such spans there is considerable ARCHING EFFECT, the concrete will safely dist the load on larger spans than those given in the table, provided a suf number of tie-rods of proper size are used to take up the THRUST of the arch

Table IX. The Limiting Spacing for Steel Beams Used With Concrete Fi

Depths of beams	Spacing of beams for the following pressures per square foot											
	1 ton		2 tons		3 tons		4 tons		5 tons		6 to	
in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	
6	1	3	0	11	0	10	0	9	0	8	0	
7	1	6	1	1	0	11	0	10	0	9	0	
8	1	8	1	3	1	1	0	11	0	10	0	
9	1	11	1	5	1	2	1	0	0	11	0	
10	2	1	1	6	1	4	1	2	1	1	1	
12	2	5	1	10	1	6	1	4	1	3	1	
15	3	0	2	3	1	10	1	8	1	6	1	
18	3	8	2	8	2	3	1	11	1	9	1	
20	4	0	2	11	2	5	2	2	1	11	1	
24	4	9	3	6	2	11	2	7	2	4	2	

The Design of a Wall-Footing of steel beams is illustrated by the foll ing example: A 24-in wall carries 42 000 lb per lin ft. What should be the and spacing of steel beams to distribute the load over the foundation-bed 3 600 lb per sq ft? The required width of the footing is 42 000/3 600 = 11 8 in and the bending moment by Equation (3) is 556 800 in-lb per lin ft wall. The amount of shear, by the formula given on page 170, is $S = W(l -$ or 34 800 lb. As the beams are in double shear the single shear per linear f of wall is 17 400 lb. The required section-modulus per linear foot of wall is tained by dividing the bending moment by the allowed fiber-stress in the st or 556 800/16 000 (assumed fiber-stress) = 34.8. By referring to Table page 355, giving the section-moduli of steel beams we find that a 12-in 31½ beam has a section-modulus of 36. To satisfy the condition of bending, beams must not be spaced more than 36/34.8 = 1.03 ft, center to center. satisfy the condition of web-crippling due to direct compression, the unit co pressive stress must not exceed the value of S_b, Table II, page 575, whi for a 12-in 31½-lb beam, is 13 060 lb per sq in. The area of the beam resist compression is the length over which the load is distributed, times the w thickness. Some authorities consider that the load is distributed over a len

the loaded portion of the beam plus one-half the depth of the beam,
is and the following example the length of only the loaded portion is
n this case the area is therefore $24 \times 0.35 = 8.4$ sq in. If the beams
d 1.03 ft on centers the unit direct compression is $42\,000 \times 1.03/8.4 =$
which is well within the allowed stress given by Table II, page 575.
the condition of web-crippling due to shear, the shearing-stress must
d the value as derived from the formula for allowable shear. (See,
er XV, paragraphs and foot-notes relating to Buckling of Beam-Webs
e illustrative Example 15 in that chapter.) The approximate, allowed,
ring value may be obtained by dividing the value of S_b (Table II,
) by the factor F, the values of which are given in Table IXa,
For example, for a 12-in 31½-lb beam this shearing value = 13 060/
15 lb per sq in. The shearing capacity of the beam is obtained by
ng this unit stress by the depth of beam times the web-thickness, or
$2 \times 0.35 = 33\,240$ lb, or much more than required. Only one of the
s of web-crippling need be considered by applying the following rule:
ear divided by the depth of the beam is greater than the total load
y the product of the distance (over which the load is distributed) by
F, investigate for shear; if otherwise, investigate for direct compres-
is rule may also be expressed as follows: According as $(l - w)/l$ is greater
an $2\,D/w'F$, investigate for shear or for compression. Here l = length
w = loaded portion of beam, D = depth of beam, w' = length of beam
h the load is assumed to be distributed (often taken = $w + \frac{1}{2}D$) and
actor for the given beam obtained from Table IXa. All dimensions
taken in the same unit. If, instead of the 12-in beams, 15-in 42-lb
ving a section-modulus of 58.9 are used, the spacing will be 58.9/34.8
nearly, say 1 ft 8 in. By referring to Table IX, page 182, it is seen
pacing of the beams is well within the safe limit of the concrete and no
re necessary.

IXa. Values of Factor F* for Shearing Values for Various Beams

Beams	For standard-weight beams	For heavy-weight beams
12-in beam	1.65	1.52
15-in beam	1.71	1.50
18-in beam	1.76	1.58
20-in beam	1.77	1.62
24-in beam	1.91	1.67

ctors, F, which have been deduced to be used in connection with S_b, Table II,
5, to give the safe unit shearing value based on web-crippling, will help greatly
ations of shears in case tables of safe shears are not obtainable. It is to be
ever, that the values derived from the use of F are approximate only, as this
little different for every beam; and to give its value for every beam would
much space as complete tables of safe shears. The values of F are not given
r sections of light beams as they are not usually good sections for grillages.
mentioned that the standard weight for each size of beam for which F is given
e next weight higher than the minimum weight given in Table II, pages 574–5.
the 20-in beams, for which the minimum weight, 65 lb, is also the standard
he rule given above for determining whether web-crippling based on shear or
ompression is the determining condition eliminates one of the calculations to
investigating grillages.

the given conditions may be designed of two or of three layers. If
are used the length of the top beams will be 11 ft. Assuming the co
= 30 in, the loaded portion = 2½ ft, and by Formula (1), the bending
= ¼ × 1 152 000 lb × (11 − 2½) × 12 × ½ = 14 688 000 in-lb, from
required section-modulus (at 16 000-lb maximum fiber-stress) = 918.
ferring to Table IV, Chapter X, five 24-in 90-lb beams have a sectio
of 932.5 and consequently satisfy the condition of bending. By ap
rule given in the preceding paragraph for the design of a wall-footing
web-crippling due to shear or to compression is to be investigated.
0.773 and $2\,D/w'F = 0.958$, which, being greater than 0.773, shows that
should be investigated for web-crippling due to compression, by th
explained in the previous example. It will be found that the five
beams also satisfy this condition and will therefore be used. The
width is about 7⅛ in, so they should be spaced about 9½ in on cente
ing the length of the column-base to be about 3 ft 9 in. The calcu
the lower layer is similar, the length of the beams being 15 ft and
portion, 3 ft 9 in. It is rarely necessary to investigate the lower laye
crippling, the condition of bending, except for the top layer, being
governing feature. If, owing to conditions of bending, it is not pra
make the beams of the top layer sufficiently long to extend across th
width of the concrete mat, it is then necessary to make the grillag
layers. The calculation for a three-layer grillage for the same probl
preceding is as follows:

Calculation of the Top Layer. For web-crippling due to co
1 152 000 lb = $S_b \times w' \times t \times n$, where S_b = the allowable unit stress
length of beam over which the load is assumed to be distributed, t =
thickness and n = the number of beams. Referring to Table II, Ch
and assuming a 20-in 75-lb beam to be used, S_b = 13 660 lb per sq
0.649 in. Taking w' = 30 in (the width of the column-base), 13 6
0.649 = 265 960 lb and the value for five beams is 1 329 800 lb, whic
than enough. But it is found that five 20-in 70-lb beams would no
cient. It will be economical to make these beams of the greatest length
they will resist bending. The section-modulus of one beam is 126.9
total M = 5 × 126.9 × 16 000 (assumed fiber-stress). This may be d

of the flanges of these beams is nearly 6½ in, so that they should
om 8½ to 9 in, thus making the required length of column-base
in.

of the Second Layer. Since the length of the top layer is limited
nd the width of the lowest layer is 11 ft, it will be necessary to have
liate layer. This
he area given by
beams of the top
the width of the
or 8 ft 4 in by 11
ams will of course
angles to those of
er, so their length
t, and they are to
as not to exceed
Since the width of
rse is 3½ ft, their
(11 ft − 3½ ft)/2
e amount of single
52 000 × 3.75/11 =
and the bending
4 × 1 152 000 × 45
000 in-lb. Using
the fiber-stress the
ction-modulus is
eferring to Table
r X, for section-
determining the
hear as above ex-
find that ten 15-in
will have a total
ulus of 812, and
lso be ample for
rthermore, ten
d to cover a width
will give a spacing,
nter of beams, of
which is sufficient.
e better, however,
18-in 55-lb beams.

of the Bottom
ing the effective
e middle layer as
projection of the
ft − 8 ft)/2 = 3½
similarly to the
hear = 268 800 lb.
6 000 in-lb, from
ction-modulus = 756, and thirteen 15-in 42-lb beams, spaced 10½
s, will be required, or two 15-in 60-lb beams and ten 15-in 42-lb
be used, increasing the spacing between the beams. In this case
eams should be placed nearest to the center of the footing. This
ustrated in Fig. 29.

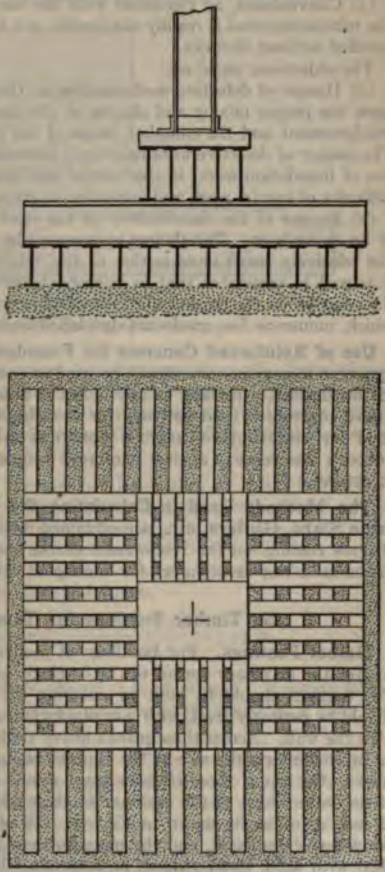

Fig. 29. Steel-beam Grillage Column-footing

upon the proper mixing and placing of the concrete, the proper pla
reinforcement and the complete union of the concrete with the rein
The danger of defective workmanship is increased by reason of the usu
ties of foundation-work, in that water and mud are generally prese
difficulty of careful work and inspection is greater.

(2) Danger of the deterioration of the steel reinforcement either b
or by electrolysis. This danger is increased by the presence of moistu
the relatively small cross-section of the reinforcing-bars. In this c
it is well to remember that in reinforced-concrete girders as usually
the concrete on the tension side is stressed beyond its elastic limit, as a
which, numerous fine cracks are developed under the figured load.

Use of Reinforced Concrete for Foundations. From the foreg
apparent that great care should be used in connection with reinforced
in foundations, especially as any defect is difficult to detect or repai
forced concrete is used not only for so-called MATS or SLABS but is f
used for DISTRIBUTING-GIRDERS, BOLSTERS and even for CANTILEVE
author's preference is against reinforced concrete for foundations for i
structures.

**The Methods Used in Calculating the Strength of Reinforc
crete Slabs, Girders, etc.,** are explained in Chapters XXIV and XX
stresses coming on the reinforced-concrete construction are to be d
in the same way as explained for footings of other materials.

25. Timber Footings for Temporary Buildings

Timber Footings. For buildings of moderate height timber may
to give the necessary spread to the footings, provided water is alway
The footings should be built by covering the bottom of the trench
should be perfectly level, with 2-in planks laid close together and longi
with the wall. Across these planks heavy timbers should be laid, spac
12 in on centers, the size of the timbers being proportioned to the t
stress. On top of these timbers again should be spiked a floor of 3-
of the same width as the masonry footings which are laid upon it.
of such a footing is shown in Fig. 30. All of the timber-work mus

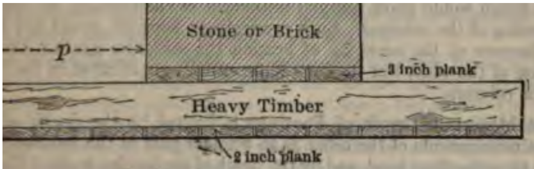

Fig. 30. Spread Footing of Timber

. The side walls of a given building impose on the foundation a f 20 000 lb per lin ft; the soil will only support, without excessive 2 000 lb per sq ft. It is decided for economy to build the footings as 'ig. 30, using long-leaf yellow-pine timber. What should be the size sverse timbers?

. Dividing the total pressure per linear foot by 2 000 lb, we have e width of the footings. The masonry footing we will make of granite ard stone, 4 ft wide, and solidly bedded on the planks in Portland, ortar. The projection p of the transverse beams will then be 3 ft. ace the beams 12 in on centers, so that $s = 1$, and will assume 10 in th of the beams. Then, by the formula,

$$\text{the breadth in inches} = \frac{2 \times 2000 \times 9 \times 1}{100 \times 67} = 5.37 \text{ in}$$

uld use 6 by 10-in timbers, spaced 12 in on centers. If spruce timber we should substitute 39 for 67, and the result would be a little over

tions for Temporary Buildings. When temporary buildings are on a compressible soil, the foundations may, in some parts of the e constructed more cheaply of timber than of any other material, and es the durability of the timber need not be considered, as when it is

has even been proposed to design the bases of the columns with ball-
joints which would allow unequal settlement of the footings witho
or bending of the columns. Such connections, however, could not
used because of the necessity of bracing the structure against th
pressure of the wind, but they would be entirely practicable in the
interior columns.

The Minimum Depth of Footings is limited by the depth o
by the requirements of the cellar as to whether part of the footings
above the cellar-floor level, and by the depth of the footing itself. Tl
depth will be advantageously exceeded if, by a slight increase in deptl
capable of sustaining a higher unit load is found on which to rest t
or if, as explained in previous articles of this chapter, greater securit
by locating the footing at a greater depth. These considerations w
the design of a footing and in all cases should be taken into conside
some cases it may be cheaper to abandon the use of a spread footing
and resort to piles or masonry construction going to rock or to
solid substratum. Where there is any question on this point, careful
should be made of the advantages and costs of the two methods.
however, it will be cheaper to spread footings immediately below
excavation level than to employ any of the various deep-foundation

Deep Foundations are necessary when the material at the lev
spread footings would ordinarily be constructed is not suitable, o
is desirable for any reason to carry the foundations of the building
underlying stratum of greater supporting power. Recourse must t
to one or more of the following expedients:

(1) Wooden piles;
(2) Concrete piles;
(3) Piers or walls constructed in pits or trenches, or by other m
going down to the required depth to reach a solid stratum.

27. Wooden-Pile Foundations

The Use of Wooden Piles. When it is required to build l **Requirements.** The laws of Boston require that wooden piles pped with block-granite levelers or with Portland-cement concrete, spacing shall not exceed 3 ft between centers. The laws of Chicago wooden piles shall be driven to rock or hard-pan and capped with imber, concrete, or steel, or a combination of these. The laws of pecify a minimum diameter of 5 inches and a maximum spacing of een centers.

ximum **Loads Allowed on Wooden Piles** in various cities are as lanta, 20 tons; Philadelphia, 20 tons; Buffalo, 25 tons; Minneapolis, ichmond, 25 tons; St. Louis, as many tons as the piles will safely hicago, 25 tons; Louisville, 20 tons; St. Paul, 25 tons; New York, rtland, Ore., 25 tons; Cleveland, 25 tons. Most of the above cities he allowed load by Wellington's formula which is hereinafter given , under the heading, Bearing-Power of Piles.

f **Wood Used for Piles.** Wooden piles are made from the trunks should be as straight as possible and not less than 5 in in diameter l end for light buildings, or 8 in for heavy buildings. The woods ed for piles are spruce, hemlock, white pine, Norway pine, long-leaf af yellow pine, pitch-pine, cypress, Douglas fir, and occasionally oak, n, black gum and basswood. There does not appear to be much n the woods as to nder water, but the stronger woods are red, especially where re to be driven to d heavily loaded.

g **Wooden Piles** g. The piles should ED FOR DRIVING by ll limbs close to the sawing the ends is probably better the bark, although re often driven with , and it is doubtful if kes much difference or the other. For soft and silty soils, has shown that the better with a square en the penetration is in at each blow the pile should be pro- BROOMING by put- RON RING, about 1 in eter than the head of d from 2½ to 3 in in thick. The head should be chamfered to fit the ring. When compact soil, such as sand, gravel, or stiff clay, the point of uld be SHOD with iron or steel. The method shown at *A*, Fig. 31, y well for all but very hard soils, and for these a CAST CONICAL POINT n diameter, secured by a long DOWEL, with a RING around the end s shown at *B*, makes the best shoe. Piles that are to be driven in or

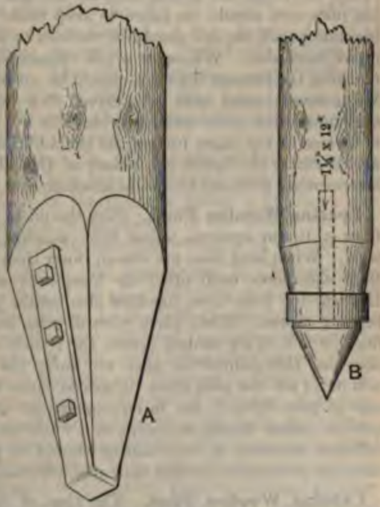

Fig. 31. Points of Wooden Piles Prepared for Driving

driven with the small end down. The hammer is generally raised by s
and is dropped either automatically or by hand. The usual we
hammers used for driving piles for building foundations is from 1 500
and the fall varies from 5 to 20 ft, the last blows being given with
Occasionally, hammers weighing up to 4 000 pounds and over are us

Driving Wooden Piles with the Steam-Hammer. Steam
are to a considerable extent taking the place of the ordinary drop-
large cities, as they will drive many more piles in a day, and with
to the piles. The steam-hammer delivers short, quick blows, fro
seventy to the minute, and seems to jar the piles down, the short
tween the blows not giving time for the soil to settle around them.
ing piles care should be taken to keep them plumb, and when the
becomes small, the fall should be reduced to about 5 ft, the blows be
rapid succession. Whenever a pile refuses to sink under several bl
reaching the average depth, it should be cut off and another pile driv
When several piles have been driven to a depth of 20 ft or more a
sink more than ½ in under five blows of a 1 200-pound hammer fall
is useless to try them further, as the additional blows only result
and crushing the heads and points of the piles, and splitting and c
intermediate portions to an unknown extent.

Spacing Wooden Piles. Piles should be spaced not less than 2
than 3 ft, on centers, unless iron, wooden, or reinforced-concrete
used. When long piles are driven closer than 2 ft on centers there is
they may force each other up from their solid bed on the beari
Driving the piles close together also breaks up the ground and di
bearing power. When three rows of piles are used the most satisf
ing is 2 ft 6 in on centers across the trench and 3 ft on centers lo
provided this number of piles will carry the weight of the buildi
will not, then the piles must be spaced closer together longitudinally,
row of piles driven; but in no case should the piles be less than 2
centers, unless driven by means of a water-jet. The number of pile
different portions of the building should be proportioned to the w
they are to support, so that each pile will receive very nearly the sa

cement mortar on the capping. Fig.
partial piling-plan, with the arrange-
the capping stones, of the Boston
of Commerce Building. It may be
nost of the stones rest on three piles,
few on two piles.

Capping. In New York City a very
ethod of capping is to excavate to a
ft below the tops of the piles and 1 ft
them and to fill the space thus
solid with Portland-cement concrete,
layers and well rammed. After the
brought level with the tops of the
ional layers are deposited over the
of the foundation until the concrete
epth of 18 in above the piles. On this foundation brick or stone
laid as on solid earth. If long bars of twisted steel, about ¾ in
ross-section are embedded in the concrete about 3 in above the
piles, the construction makes, in the opinion of the author, the best
ping, the twisted bars giving great transverse strength to the concrete.

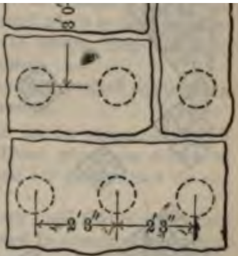

Fig. 32. Stone Capping for Three Rows of Wooden Piles

rillage Capping. Most of the pile foundations of Chicago have
er grillages bolted to the tops of the piles and stone or concrete foot-
top of the grillages. The timbers for the grillages should be at least
in cross-section, and should have sufficient transverse strength to
load from center to center of piles, using a low fiber-stress. They
id longitudinally on top of the piles and fastened to them by means
LTS, which are plain bars of iron, either round or square in section,
into holes about 20% smaller in section than the bolts themselves.
quare bars 1 in in section are generally used, the holes being bored
uger for the round bolts and by a ⅞-in auger for the square bolts.
ould enter the piles at least 1 ft. If heavy stone or concrete footings
l the space between the piles and timbers is filled with concrete level

The Advantages of Timber Grillage are that it is easily laid and
holds the tops of the piles in place. It also tends to distribute th
evenly over the piles, as the transverse strength of the timber will he

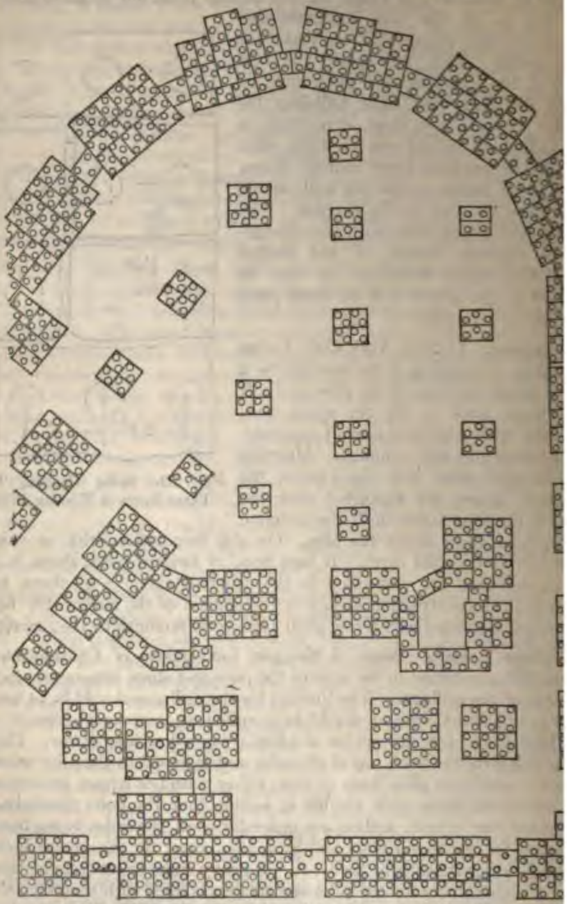

Fig. 33. Piling-plan, Chamber of Commerce Building, Boston, Ma

the load over a single pile, which for some reason may not have the sa
capacity as the others. Steel beams, embedded in concrete, are some
to distribute the weight over piles, but some other form of constr
generally be employed at less expense and with equally good results.

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* For a description of the pile foundations and capping of the Chicago Po
Freitag's Architectural Engineering, pages 350 to 352.

o not move more than 3 in under the last five blows of a hammer ooo lb and falling 20 ft. All split or shattered piles are to be re- ossible and a good one driven in place of each imperfect one. In such piles cannot be removed an additional pile is to be driven for fect one. If the piles show a tendency to BROOM, they are to be wrought-iron rings, 2½ in wide and ½ in thick.

when driven to the required depth, are to be sawed off square for a earing at the grade indicated on the drawings.

ring Power of Piles. In regard to their use for supporting build- may be divided into two classes: (1) Those which are driven to RD-PAN, that is, firm GRAVEL or CLAY and (2) those which do not -PAN.

e belonging to this class when driven through a soil that is sufficiently e the pile at every point, may be computed to sustain a load equal resistance to crushing on the least cross-section. If the surrounding ic the bearing power of the pile will be its safe load computed as a ving a length equal to the length of the pile when capped. Test- on the site of the Chicago Public Library Building, through 27 ft stic clay, 23 ft of tough, compact clay and 2 ft into hard-pan, sus- d of 50.7 tons per pile for two weeks without apparent settlement. nany instances where piles driven to the depth of 20 ft in hard clay n 20 to 40 tons, and a few instances where they sustain up to 80 tons

le belonging to this class depends for its bearing power upon the OHESION and BUOYANCY of the soil into which it is driven. The or such piles is usually determined by the average penetration of der the last four or five blows of the hammer. Several engineers lated rules for determining the safe loads for piles of this class, but many conditions that modify the amount of the penetration, and etermination, and so many varying conditions of driving and of is considered impossible to formulate any rule that can be considered isfactory for all the conditions under which such piles are driven.

neering News Formula. The formula generally used by engineers

new calculations must be made based on the smallest average penet
can be obtained, and a greater number of piles used. In localities w
is commonly used for foundations, the least penetration that can b
within practical limits of length of pile can generally be ascertained b
tion, or by consulting somebody who is experienced in driving piles.
the pile the less, as a rule, will be the final SET or penetration. Where
experience to guide one it will be necessary to drive a few piles to
the length of pile required, or the least SET for a given length of p
piles will have to be driven further than others to bring them to
equal resistance. When the piles are to be loaded to more than
assumed safe load, the final set of each pile should be carefully meas
inspector, the BROOM and SPLINTERS being removed from the head
for the last blow.

Safe Loads for Piles. Table X, computed by the above formula,
safe loads for different penetrations, under different falls of a hamme
1 ton. FOR A HAMMER OF DIFFERENT WEIGHT multiply the safe load i
by the actual weight of the hammer in tons. Thus, for a hamme
1 000 lb, the values in the table should be multiplied by ½ and for
hammer, by ¾.

Table X. Safe Loads in Tons for Piles

For hammer weighing 1 ton

Penetration of pile in inches	Height of the fall of the hammer in feet										
	3	4	5	6	8	10	12	14	16	18	20
0.25	4.8	6.4	8.1	9.7	12.9	16.1	19.4	22.5	25.8	29.1	32.3
0.50	4.0	5.3	6.7	8.0	10.7	13.3	16.1	18.7	21.3	24.0	26.6
0.75	3.4	4.6	5.7	6.9	9.2	11.5	13.8	16.1	18.4	20.7	23.0
1.00	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0
1.25	..	3.6	4.5	5.4	7.1	8.9	10.7	12.5	14.3	16.1	17.9
1.50	..	3.2	4.0	4.8	6.4	8.0	9.6	11.2	12.8	14.4	16.0
1.75	..	..	3.6	4.4	5.8	7.3	8.8	10.2	11.7	13.1	14.6
2.00	..	..	3.3	4.0	5.3	6.7	8.0	9.3	10.7	12.0	13.3
2.50	..	..	..	3.4	4.6	5.7	6.9	8.0	9.1	10.3	11.4
3.00	..	..	..	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
3.50	..	..	..	..	3.6	4.4	5.3	6.2	7.1	8.0	8.9
4.00	..	..	..	..	3.2	4.0	4.8	5.6	6.4	7.2	8.0
5.00	..	..	..	..	...	3.3	4.0	4.7	5.3	6.0	6.7
6.00	..	..	..	..	...	...	3.4	4.0	4.6	5.1	5.7

Example of Computations for Pile Foundation. Suppose that
observations of the pile-driving for an adjacent building it is found
driven from 20 to 30 ft take a set of 1 in under a 1 200-lb hammer fa
and that additional blows result in about the same set.

From Table X we find that the safe load for a fall of 20 ft and
tion of 1 in is 20 tons. Multiplying by the weight of the hammer in
we have 12 tons as the safe load per pile. Suppose that the total
lin ft of footing is 13 tons. As we must have at least two rows of
each two piles will support 24 tons, it follows that the spacing of
longitudinally should be 24/13 = 1 ft 10 in. As this is too close, we
three rows of piles, spaced 2 ft apart laterally, and the longitudin

in diameter at the butt and 6 in at the point, driven 31 ft into hard,
y near Haymarket Square, failed to show movement under 30 tons, nate load being probably 60 tons.* Other piles driven 17.9 ft sustained f 31 tons each. The average penetration under the last ten blows of a hammer falling from 9 to 12 ft varied from 0.4 to 0.95 in per blow for iles.

5 ft long under the Chamber of Commerce Building penetrated about er the last blow of a 2 000-lb hammer falling about 15 ft.

GO. In the Public Library Building the piles were proportioned to each and were tested to 50.7 tons without settlement.

Schiller Building the estimated load was 55 tons per pile; the building rom 1½ to 2¼ in.

Passenger Station of the Northern Pacific Railroad, at Harrison Street, ft long were designed to carry 25 tons each and did so without per- settlement.

rt Institute Building, parts of the Stock Exchange Building and also number of warehouses and other buildings on the banks of the river piles.

YORK CITY. The Ivins (Park Row) Building is supported by about -in spruce piles, arranged in clusters of fifty or sixty, for single columns, orresponding number under piers supporting two or more columns. s were driven to refusal of 1 in under a 20-ft fall of a 2 000-lb hammer. erial is fine, dense sand to a depth of over 90 ft. But few piles could be nore than 15 or 20 ft. The average maximum load per pile is 9 tons.† merican Tract Society's Building is supported on piles.

LYN. Piles under the Government Graving Dock, driven 32 ft, on age, into fine sand mixed with fine mica and a little vegetable loam, are l to sustain from 10 to 15 tons each.

ORLEANS. Piles driven from 25 to 40 ft into a soft alluvial soil carry om 15 to 25 tons, with a factor of safety of from 6 to 8.‡

ost of Driving Wooden Piles. The cost of driving piles naturally ith the character of the soil, and the conditions under which they are

by Horace J. Howe, was published in the American Architect and Buil
commencing June 11, 1898. The paper records a great many tests
several formulas and many experiences of distinguished engineers.
Building Construction and Superintendence, by F. E. Kidder, gives
information in regard to pile foundations and experiments on the bea
of piles. Much valuable information on piles is given in "A Practica
on Foundations," by W. M. Patton. The recent Engineers' Handb
should be consulted for additional data.

28. Concrete-Pile Foundations

Durability of Wooden and Concrete Piles. Concrete piles, ei
or reinforced, possess many advantages over wooden piles and, in ge
be used in all places where wooden piles can be driven. Concrete p
pared with wooden piles, have primarily the advantage of greater PER
Timber piles, kept constantly wet and protected from the action of the
or other destructive influences, may be practically everlasting, but
counted upon above water level; whereas concrete piles should be pro
all deteriorating actions, whether wet or dry, except the action of f
wet concrete.

Strength of Wooden and Concrete Piles. Concrete piles wit
forcement, if made of good concrete, should have nearly the same
STRENGTH per square inch as ordinary yellow-pine piles, and with
placed reinforcement concrete piles should have a much higher crushin
per square inch than timber piles. Moreover, timber piles do not
FORM CROSS-SECTIONS. For instance, a slender timber pile 40 ft in l
12 in in diameter at the butt, is probably not over 6 in in diameter at
In direct compression the load on a point-bearing pile of the above d
is limited to the safe load on the point of the pile, where it is 6 in in
and a cylindrical concrete pile, 12 in in diameter and under similar c
will have a cross-section of 113 sq in at all points, compared with
section of 28 sq in at the point of the timber pile. Moreover, if w
both piles as LONG COLUMNS, it must be borne in mind that a timber pil
be straight and that it may, therefore, be subject to STRESSES and DEF

any one of several different methods.

aymond Method of Forming Concrete Piles in Place. In the
RAYMOND PILE METHOD a steel MANDREL of tapered form is driven into
d, and when the required penetration has been obtained this mandrel
ed and withdrawn, leaving a hole corresponding to the size of the ex-
andrel in the ground; this hole is then filled with concrete. The rein-
may be placed in the hole prior to the placement of the concrete.
hod, as described, is applicable only to such material as will stand
mandrel is being withdrawn and the hole is being filled with concrete.
ases the method used is as here described except that a thin shell of
aced on the mandrel before driving. When the mandrel is collapsed
is left in the ground, thus forming a lining for the hole which is sub-
filled with concrete or with reinforcing-rods and concrete, as before
Raymond piles have been extensively used, especially for FRICTION-
SKIN-BEARING PILES in soft and artificially filled-in ground. An im-
rm of lining-shell recently employed in the Raymond method is com-
h a spiral reinforcement inside of the shell, which materially assists in
g the collapse of the shell.

mplex Method of Forming Concrete Piles in Place. THE SIMPLEX
differs from the Raymond method and may be briefly described as
A steel pipe, generally cylindrical in form, of the required size and
d fitted with a detachable cast-iron conical DRIVING-POINT, is driven into
d to the required depth; the pipe is then partially filled with concrete.
-like PLUNGER, smaller in diameter than the inside diameter of the
hen placed on the concrete and the pipe is partially withdrawn, leaving
ng-point and part of the superimposed concrete in the ground. This
is repeated until the pile is built up to the required height. In cer-
erials, instead of using a detachable driving-point, the driving-point
f two jaws hinged to the lower end of the pipe, so arranged that while
e driving they form a driving-point, when the pipe is withdrawn they
form an extension of the cylindrical pipe. In other words, the jaws
ed of steel plates previously bent to the same radius as the radius of

Composite Piles. Protected piles, for use in localities where the
affects the life of timber piles under water, are composed of timber p
concrete coatings held in position by steel reinforcements in the shape of e
metal or wire netting. Such piles are to be considered as timber pil
than as concrete piles.

Timber Piles with Concrete Caps. In some localities where the pe
water-level is considerably below the level of the required excavatio
piles have been driven with a FOLLOWER, the follower consisting of a s
or cylindrical shell. When the head of the pile is driven to a safe
below low water the PIPE-FOLLOWER is filled with concrete and wit
leaving the concrete pier resting on a timber pile. This composite pil
appear to possess the advantage of combining the cheapness of a tim
below the water-level with the permanency of a concrete pile above th
level. Great care, however, should be used in adopting this method on
of the difficulty of securing proper connection between the concrete
wooden pile.

The Methods used in Driving Built-up Piles are practically t
as are used in driving wooden piles, except that a CUSHION of wood,
other material is placed on the head of the pile to be driven to cushion
of the hammer. Steam-driven or air-driven RECIPROCATING HAMMERS
erable to the ordinary DROP-HAMMERS. In stiff materials the use of a
JET is advisable and, in fact, in many cases indispensable. In lifting
piles use is made of a special SLING which is attached to a pile at two poi
point one-quarter of the length of the pile from the end. The sling sho
a SPREADER so that the stress due to the oblique pull of the CHAIN-
taken up by the spreader rather than by the pile.

The Casting of Concrete Piles. Concrete piles should be CAST
PIECE by a continuous operation so that there will be no PLANE OF W
formed between partially set concrete and fresh concrete. They may
either in a vertical position, in forms, or in a horizontal position. Sq
tion concrete piles have been cast in a horizontal position and side-for
used, the previously cast concrete pile, protected by paper, forming th
form. In some cases, where it is intended to use a WATER-JET in sinki

depth; whereas, if four concrete piles, placed 3 ft from center
l, instead of wooden piles, the area of the base will be a little
nd the projection will be only 1 ft. A suitable footing would
rced-concrete cap not over 2 ft in thickness. The saving in
, concrete and steel in the footing is all in favor of the use

under Walls. In the case of a continuous wall, where the
t of wall is not great, a single row of concrete piles is often
rt the weight of the wall. In such cases, the piles should
raight lines but should be STAGGERED, and a sufficient footing
ted connecting the heads of the piles, so as to afford stability

Employed in Calculating Reinforcement for Concrete
as that employed in calculating ordinary reinforced-concrete
difference being that where a pile is not point-bearing, but is
surrounding material for its support, it need not be considered
N. POINT-BEARING PILES deriving their support from some
which their lower extremity rests, must be considered as LONG
assumption that the material surrounding the piles may fail
In the case of FRICTION-PILES, depending for their support
ling material, this assumption cannot be made, as any failure
l involve a settlement of the pile. It should be borne in mind
supported on piles supported by SKIN-FRICTION is dependent
on the continued supporting power of the material surround-
many cases buildings resting on piles driven into soft ground
e result of the consolidation and settlement of the material
iles, notwithstanding the fact that the piles when driven were
port the loads for which they were designed.

s with Concrete or Reinforced-Concrete Filling have been
wooden or concrete piles, especially in UNDERPINNING-WORK.
he use of such piles is that the iron pipe forming the external
which case the strength of the pile is reduced to the strength

Foundation Piers and Walls as distinguished from ordinary CELL and WALLS, extend from the level of the underside of the cellar-floor t other solid foundation-bed. (See page 129, Subdivision 1, and also III, pages 228–9.) In general, such piers and walls are composed of and are of such dimensions that the safe unit loads on the concrete them are not exceeded. If the foundation-bed is rock, compact har gravel, there need be little or no enlargement of the base of the pier o the safe unit loads on such natural foundation-beds are generally equ safe unit loads on the concrete forming the body of the pier or wall. T of such piers and walls is therefore an entirely simple matter governe principles already outlined, and by certain considerations mentioned

The Methods used in the Construction of Foundation P Walls are, however, necessarily varied to suit different materials an different conditions encountered, and the design of a pier necessarily di different methods of construction. For example, if the constructio executed by means of the ordinary SHEET-PILING METHOD, piers and have in general rectangular outlines. But if the CHICAGO METHOD or MATIC CAISSON is employed, it will generally be cheaper to use piers circular cross-section and the support for walls may be a succession of rather than continuous walls. The detailing of the concrete structu tuting the piers or walls is simple after a determination is made of th by which the construction is to be put in place. This subject is discus following chapter-subdivision, Methods of Excavating for Foundation

30. Methods of Excavating for Foundations

Simple and Complex Excavations. Excavations in earth for f walls and piers may vary from simple trenches and pits of the req and depths to accommodate the footings, up to deep subaqueous e requiring all the resources of engineering skill.

The Sides of Excavations. If the earth is firm and the depth no the sides of the excavation may be self-supporting, in which case the may be made the neat size of the footing and the sides of the excav ill not stand with a e, even if braced in r, for any length of if the material is

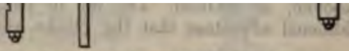

Fig. 34. Extensible Brace for Narrow Excavations

or soft clay, such bracing is entirely inadequate. In such cases, ct generally, some form of CONTINUOUS SHEET-PILING must be

y Wooden Sheet-Piling consists of a continuous line of vertical l against the sides of the excavation by horizontal timbers known as LING or BREAST-TIMBERS, these wales, or breast-timbers being in turn either by CROSS-BRACES extending across the excavation to an oppo- r side of the excavation, or by inclined struts known as SHORES or xtending to the bottom of the excavation where HEELS or inclined are sunk in the undisturbed material to afford points of support.

ressure on Sheet-Piling. The load on the sheeting due to the SSURE may be calculated on the assumptions made for the design of -WALLS, but the thickness of the sheeting planks, the sizes and spacing st-pieces and braces, if figured on this basis, will in general exceed the antly used with success and safety in such work. The probable reason hat an earth bank, when steadied and in part supported by the sheet- ot, for a considerable time, lose the COHESION between its particles most earth banks in their original and undisturbed state. Or, in s, under these conditions no real ANGLE OF FRICTION is developed in mass. Local experience and practice should be consulted and will erve as a guide. Earth banks apparently similar will, however, act ently and no general rule can be given. It should be borne in mind rth composing a bank should be, as far as possible, protected from he action of water and from alternating freezing and thawing; and anent work should be completed as rapidly as possible so as to deteriorating effects of time and exposure on the structure of the

ickness of the Sheeting Planks required may be calculated on the that the earth bank is composed of loose material having a definite SLOPE and COEFFICIENT OF FRICTION; but practically, under favorable the planks in the second set or DRIVE have to be placed inside of the lo pieces, thus reducing the size of the excavation by the amount of the seco offset. Where more than one drive is required the first drive should be sta at a sufficient distance outside to allow the planks forming the second or second and third drives to be placed outside of the required area for the b tom of the excavation.

Cutting and Fitting Sheeting Planks. The sheeting planks may SQUARE-EDGED where there is no water or fine loose sand, but where wat or running sand is to be excluded the planks should be TONGUED GROOVED, or SPLINED. The use of tongued and grooved planks has additional advantage that the planks are more readily kept in line. It

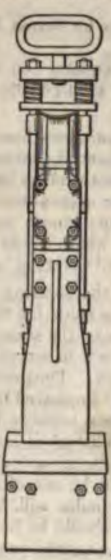

Fig. 35. Small Power-hammer for Driving Sheeting Planks

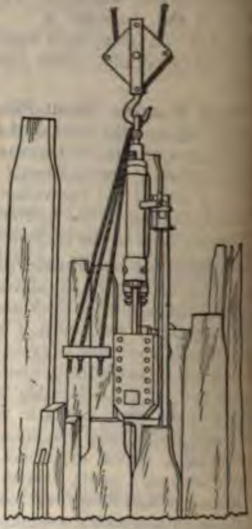

Fig. 36. Large-size Power-hammer and Sheeting Planks

usual to cut the bottom edge of each plank on a slight angle, so that in drivi it is WEDGED against the preceding plank. The top of each plank may be fitte to receive an iron DRIVING-CAP; or, if this is not used, the upper corners of th plank should be cut off so that the effect of the blows will be concentrate along its vertical axis, and the tendency of the plank to SPLIT, due to a blo on one corner, thus diminished.

The Means Employed for Driving the Sheeting vary with the dept and the size of the sheeting. For small jobs and for moderate depths drive, the primitive method of DRIVING BY HAND with ringed wooden MAU still prevails. For work involving a considerable amount of driving, and in a cases for long drives, POWER-HAMMERS driven either by steam or compressed ai are preferably employed. A small-sized power-hammer (Fig. 35) resembles STEAM-DRILL and may be handled by two or three men without any special liftin

The larger sizes of power-hammers (Figs. 36 and 37) are practically pile-driving hammers arranged with a special DRIVING-HEAD to ıg employed. Such hammers are handled by DERRICKS or are frame similar to a pile-driver frame. Ordinary DROP-HAMMERS used, but are not as advantageous as the RECIPROCATING POWER- he blow struck by the drop-hammer lank, while the frequent light blows ammer tend to keep the planks and aterial in motion and accomplish the with less damage to the sheet-piling. nd dimensions of several types of pile- ers are given in Table XI, page 204.

Driving Sheeting Piles. In prac- w excavation is first made to the the outside of the sheeting planks. ST-TIMBER is temporarily secured in lower end of the planks placed be- nber and the bank. If the planks orary TOP GUIDES or STAY-BRACES are s to keep the planks vertical until n driven well into the ground and e permanent BREAST-PIECES. The ıen driven as the excavation pro- plank being driven a few inches in driving goes on the material under e of the planks is loosened with a a crowbar, the operation being so t the planks are held true to line. breast-timbers and their braces are ition as the excavation progresses. ces are to be used the excavation in ken out first, leaving a sloping bank les of the excavation. This permits of the inclined braces and of the points of support before there is any bank. After the first breast-piece d brace are set in place, the second t breast-pieces and braces are put in ion proceeds.

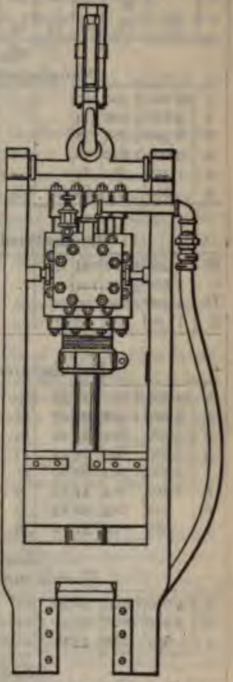

Fig. 37. Large-size Power-hammer for Driving Sheeting Planks

g for Excavations Below Water- excavations may be made by NG METHOD if there is not too much water and if water can of the material without inducing a flow of sand or clay below the sheet-piling. In some cases, where unfavorable conditions re there is an underlying stratum of impervious material, it is ve the sheeting in advance of the excavation, so that the bottom makes a tight joint with the impervious stratum, cutting off the and material. Where a considerable amount of water finds its xcavation, the water must be led to a SUMP or depression from ted by means of a PUMP or a STEAM-SYPHON. Where the founda- ow water-level and the material is sand, clay, or other material e softened by the action of the water, it should be protected by p at a considerable distance from the area to be used for the sup-

Table XI. Weights and Dimensions of Pile-Driving Hamme

Size-No.	Total net weight	Weight of ram	Dimensions over all			Cylinder			Steam-boiler H. P. required	Comp. air, free air per min	Size of hose	Distance between jaws	Width of jaws	Duty, si or pili mer w
			Height	Width	Depth	Diam.	Stroke	Strokes per min						
	lb	lb	in	in	in	in	in			cuft	in	in	in	
						Warrington Steam Pile-Hammers Manufactured by Vulcan Iron Works, Chicago, Ill.								
0	16 000	7 500	180	....	....	16½	48	60	60	....	2½	26	9¼	Hvy co
1	9 850	5 000	144	....	....	13½	42	65	40	....	2	20	8¼	18" sq o
2	6 500	3 000	138	....	....	10½	36	70	25	....	1½	19	7¼	14" sq o
3	3 800	1 800	96	....	....	8	30	80	18	....	1¼	18	6¼	10" sq o
4	1 350	550	84	....	....	4	24	80	8	....	1	14	4¼	4"×12"
5	800	....	68	10	10	4	7½	300	10	....	1	....	....	3"×12"
						Cram Steam Pile-Hammers Manufactured by A. F. Bartlett & Co., Saginaw, Mich.								
B	8 400	5 500	144	....	....	....	40	70	25	....	2½	27	8¼	18" sq o
C	5 500	3 090	144	....	....	....	40	70	18	....	2	20	8¼	14" sq o
D	4 200	2 250	102	....	....	....	24	80	15	....	1¾	20	8¼	10" sq o
E	1 000	430	78	....	....	....	12	80	15	....	1¼	12	5¼	4"×12"
						Union Pile-Hammers Manufactured by Union Iron Works, Hoboken, N. J.								
0	12 100	2 550	118	28	20	10½	24	100	50	750	2	28	8½	Hvy con
1	8 000	1 548	94	28	18	9½	21	110	30	600	1½	28	8½	18" sq o
2	5 500	890	81	25	15	7¼	16	130	18	300	1¼	25	6½	14" sq o
3	4 500	663	74	23	13	6¼	14	135	15	200	1¼	23	5½	10" sq o
4	2 500	363	60	20	11	5¼	12	150	10	150	1	20	4½	6"×12"
5	1 400	214	47	17	9	4¼	9	200	8	100	1	17	4½	4"×12"
6	850	129	40	14	8	3¼	7	250	5	60	¾	14	3⅜	2"×12"
7	365	70	31	10	6	2⅜	5	300	3	40	¾	10	3⅜	1"×6"
						Goubert Steel-Pile Driving-Hammer Manufactured by A. A. Goubert, New York, N. Y.								
3	5 000	1 500	76	29	17	8	14	150	50	660	2	24	8¼	18" sq o
2	3 400	800	62	24	14	6¼	10	160	25	340	1½	22	6¼	12" sq o
1	950	200	43	16	10½	4	8	200	10	150	1¼	....	...	4" shee
						New Monarch Steam Pile-Hammer Manufactured by Henry J. McCoy Co., New York, N. Y.								
1	7 000	1 500	90	24	24	9	12½	125	35	600	2	24	8¼	18" sq o
2	4 600	850	72	20	20	7¼	11	150	20	300	1½	20	8¼	14" sq o
3	2 800	450	54	18	18	4¾	7	175	15	150	1	18	8¼	6"×12"
4	800	125	48	14	14	3¼	6	250	10	65	¾	....	...	3"×12"
						McKiernan-Terry Pile-Hammers Manufactured by McKiernan-Terry Drill Co., New York, N. Y								
9	7 500	1 500	77	21	27⅝	15	12	200	60	600	2	21	6½	18" sq o
7	5 000	800	67	21	22⅝	12½	10	225	35	350	1½	21	6½	14" sq o
5	1 500	200	56	11	14⅞	7	8½	300	20	200	1¼	11	4¼	4"×12"
3	640	68	54	9	9½	3¼	5¾	300	15	150	1	9	3½	3"×12"
1	145	21	42	8	6½	2¼	3¾	500	10	100	¾	8	2⅛	2"×10"
						Ingersoll-Rand Sheet Pile-Driver Manufactured by Ingersoll-Rand Co., New York, N. Y.								
G1	1 200	200	80	11¼	11	4	7¼	300	10	110	1¼	....	...	4"×12"

eting has been largely employed recently in place of wooden sheet-
the advantage that it can be driven in advance of the excavation,
cing the likelihood of any flow of material under the sheeting. It
advantages of affording greater strength for a given thickness of
being driven to a greater depth, and in many cases of being with-
ised over again. As generally manufactured, it has the further
being INTERLOCKING, so that there is less danger of its getting out
aving openings between adjacent pieces.

se advantages have been considered by engineers in using steel
oden sheeting.

f Steel Sheeting. The fundamental idea of steel sheeting is not
-IRON SHEET-PILING was used in England as far back as 1822 and
inations of steel plates have been used in coffer-dams. The general
heeting started in this country in 1899 when Luther P. Friestedt
nental INTERLOCKING CHANNEL-BAR SECTIONS. Since that time it
o general use, and with its aid many excavations have been made
SHEET-PILING which would have been impracticable with timber

ssure on Steel Sheeting. In using STEEL SHEETING, it should
nind that the EARTH-PRESSURE coming on the steel sheeting is the
earth-pressure coming on timber sheeting, and the breast-pieces
ould be as strong as in the case of timber sheeting. Certain forms
f steel sheeting offer considerable resistance to bending due to the
-pressure. With such forms the horizontal breast-pieces may be
er apart than with ordinary timber sheeting or steel sheeting not
property; but the strength of the breast-pieces and of their braces
cient to take up the entire load coming on the sheeting, irrespective
g between such breast-pieces, for in case there is a failure in these
eeting will fail.

Forms of Steel Sheeting. Various TYPES OF STEEL SHEETING
arket. In making a selection between different forms of sheeting,

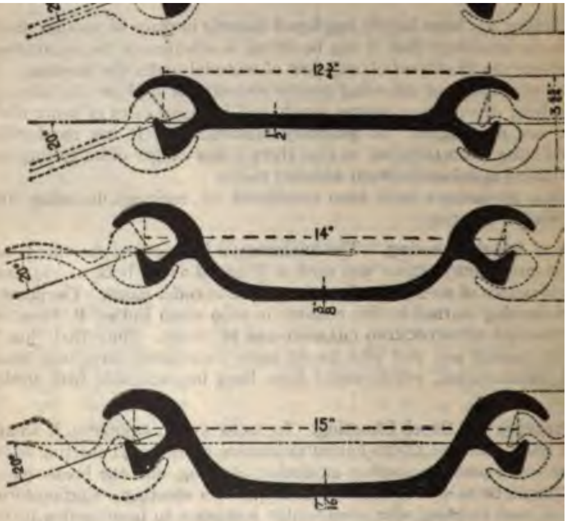

Fig. 38. Lackawanna Steel Sheet-piling

Lackawanna Steel Sheet-Piling *

Composition and Dimensions of Sections

Sections	Per linear foot, lb	Per sc

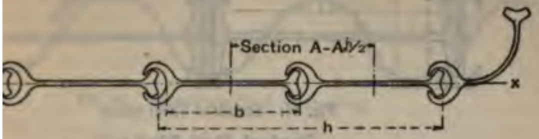

Fig. 39. United States Steel Sheet-piling

United States Steel Sheet-Piling *

Composition and Dimensions of Sections

Size	Web in	*b* in	*h*/2 in
12½ in, 38 lb..........	⅜	12½	13¼
9 in, 16 lb............	¼	9	9¼

adapted to straight or circular work.

* Manufactured by the Carnegie Steel Company.

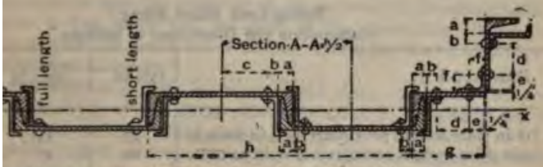

Fig. 40. Friestedt Interlocking Channel-bar Piling

Friestedt Interlocking Channel-Bar Piling *

Composition and Dimensions of Sections

Description	Channels		Zees		*h*/2, in
	In	Lbs per ft	In	Lbs per ft	
0 in, 28 lb	10	15	3⅛×¼	4.8	9
0 in, 34 lb	10	20	3⅛×¼	4.8	9
2 in, 34 lb	12	20.5	3⅝×⅜	8.6	10⅞
2 in, 39 lb	12	25	3⅝×⅜	8.6	10⅞
5 in, 39 lb	15	33	4⅛×⅜	9.2	13½
5 in, 45 lb	15	40	4⅛×⅜	9.2	13½

* Manufactured by the Carnegie Steel Company.

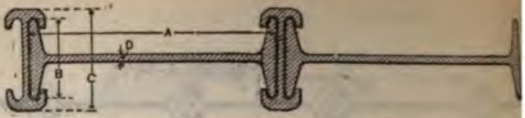

Fig. 41. Standard Sheet-piling

Standard Sheet-Piling *
Composition and Dimensions of Sections

No.	Size, in	Weight per square foot, lb	A	B	C
1	12×5	35.0	12	3.94	5
2	12×5	36.25	12	3.97	5
3	15×6	37.20	15	4.75	6
4	15×6	39.75	15	4.81	6
5	15×6	42.25	15	4.87	6

An interlocking bar is wedged to each beam at the mill and the two pieces as a unit.

* Manufactured by the Jones & Laughlin Steel Company.

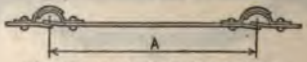

Fig. 42. Spring-lock Sheet-piling

Spring-Lock Sheet-Piling *
Composition and Dimensions of Sections

Distance, A	15¼ in	19¼ in
3/16-in plate, weight per square foot, in pounds	17	14½
¼-in plate, weight per square foot, in pounds	20	17

Plates may be obtained curved to any radius for circular work.

* Manufactured by the Mitchell-Tappen Company.

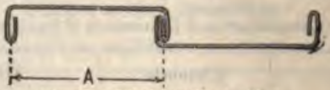

Fig. 43. Slip-joint Sheet-piling

Slip-Joint Steel Sheet-Piling *
Composition and Dimensions of Sections

Distance, A	6 in	9 in	12 in
No. 14-gauge, weight per square foot, in pounds	5.4	5.8	5.7
No. 16-gauge, weight per square foot, in pounds	4.3	4.0	3.9

* Manufactured by the Mitchell-Tappen Company.

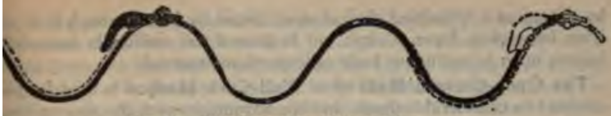

Fig. 44. Wemlinger Steel Sheet-piling

Wemlinger Steel Sheet-Piling *

Composition and Dimensions of Sections

....	1	2	3	4	5	6	7	8	9	10
of corrugation..............	2	2	2	2½	2½	2½	2½	4	4	4
ess..........................	1/16	7/64	1/8	7/64	1/8	5/32	3/16	3/16	1/4	5/16
center to center of lap......	12	12	12	12	12	12	12	18	18	18
per square foot in pounds..	5	7.5	8.5	8	9.5	11.5	13.5	15	19	23.5

mensions given are in inches.

* Manufactured by the Wemlinger Steel Piling Company.

Poling-Board or Chicago Method is a special method of excavation l use in Chicago and in occasional use elsewhere for excavations which great depth in clay or in other suitable material. It has the advanr the ordinary sheet-piling method that the lining of the excavation is en. The method is not generally used for trenches or for square exs as a circular excavation is more readily handled. The success of the depends entirely upon the character of the material to be encountered, excavation is first made and the sides of the excavation afterwards d. The method in detail for a circular excavation for a pier-founay be described as follows:

circular excavation slightly in excess of the size required for the pier d down to a depth of 5 ft, great care being taken to have the sides of vation vertical and true to the circle.

ertical planks called LAGGING-PIECES, 5 ft in length and slightly beveled edges so that each piece may be considered as a stave with radial joints nding to the size of the required circle, are set in place against the walls xcavation. These planks are held in place by two or more steel rings, y made in quadrants, so that they may be conveniently handled and ogether. The planks are wedged firmly against the walls of the excavameans of wooden wedges driven between the planks and the iron rings.

s soon as the first set of lagging is complete, the excavation is carried r another section, 5 ft in depth, and another section of lagging is put in d secured in the same manner.

and Character of Excavations in the Poling-Board Method. In the described above the excavation may be carried down for an indefinite , a depth of 100 ft having frequently been attained. In many cases tom of the excavation is BELLED OUT to a larger diameter than the ex for the main shaft of the pier with the object of reducing the load of dation-bed to a unit load less than the safe unit load on the main shaft of . This method is not adapted for running sand nor for clay that is not ough to stand with vertical sides during the necessary interval between shaft must be left in the structure to permit of the passage of men and

Details of the Open-Caisson Method. In detail, the method may be
as follows: First a CURB or CUTTING-EDGE of timber or steel, following
line of the pier, is constructed on the surface of the ground. The o
of this curb is generally vertical and is protected with a steel plate
tends below the main section of the curb, so as to form a cutting-edge
downward projection serving to penetrate the soil slightly in advan
excavation. On this curb a wall of timber, concrete, or masonry is con
inside of which the so-called WORKING-CHAMBER affords room for the
to be employed in excavating. Above the working-chamber the walls
tinue to a height corresponding to the required height of the pier, le
central space to be filled in after the required depth is reached; or a
be built over the working-chamber and the entire cross-section of the
with concrete or masonry excepting only a small central opening large
accommodate a HOISTING-TUB or BUCKET and to permit of the ingress a
of the men employed in sinking the construction. In practice, the ex
is started before the pier-structure is carried up to its final height, aft
the excavation and the building up of the pier progresses simultaneo
constantly increasing weight of the structure aiding the sinking of
When the excavation has reached rock or a firm substratum, further
tion is stopped and the working-chamber and the central opening ar
full of concrete, leaving finally a complete pier-structure extending
rock to the proper level to receive the steel grillage or other construct
ing on the pier.

Advantages of the Open-Caisson Method. This method of constru
the advantages that the workmen at all times are protected, that obs
such as boulders or logs, may be removed from under the cutting-
that when rock is encountered, ample opportunity is afforded for th
preparation of the rock-surface to receive the final concrete filling. If a
amount of water is encountered, not accompanied by a flow of mater
generally be taken care of by means of pumps.

Dredged Wells are similar to the open caissons described in the
paragraphs and are used where large quantities of water are encounte

arth temporarily back-filled against it. As soon as the four planks
e SECOND SET have been put in place by this method, the two side
wedged against the bank, the end-planks being used as struts. The
are wedged into position and nailed or cleated to the side planks
PRESSURE-RESISTING FRAME supporting the side of the excavation.
ation of this method enables the excavation to be carried on indefi-
vided there is no flow of water or run of material causing an inflow
into the excavation.

eumatic-Caisson Method. Where piers or foundation walls have
ed to a considerable depth through water-bearing materials, and espe-
e large bodies of quicksand are encountered, the PNEUMATIC-CAISSON
ust be resorted to. This method is based upon the PRINCIPLE OF A
L and may be briefly described as follows: The construction of the
ilar to the piers previously described as used in the open-caisson and
ell construction, except that the working-chamber and shaft are made
d connected with a device called an AIR-LOCK, so that compressed air
troduced into the working-chamber. The object of the compressed
event water entering into the working-chamber. This is accomplished
nce with the well-known PRINCIPLE OF THE DIVING-BELL by having
essed air constantly kept at a pressure which will counterbalance the
sure at the level of the cutting-edge of the working-chamber. The
the air evidently must vary with the depth of the cutting-edge below
. A column of water 1 in square in cross-section weighs .43⅓ lb per
and it will therefore be counterbalanced by an air-pressure of .43⅓
n over the normal air-pressure. If the column of water is 30 ft in
vill weigh thirty times .43⅓ lb, or will be counterbalanced by an air-
13 lb per sq in above the atmospheric pressure.

imum Air-Pressure in the Pneumatic Caisson in which men can work
eriods is about 43 lb per sq in above atmospheric pressure, correspond-
epth below water-level of about 100 ft. At this depth the work is
in shifts of from two to three hours duration, and great care must
d in coming out of the AIR-PRESSURE. The physiological effects of

first, to close the upper door, and secondly, to shift the so-called eq VALVE and admit compressed air into the space between the two do the air-pressure is brought up to the air-pressure in the working-cha shaft. Pressure on the upper side of the lower door will then equal th on the lower side and the lower door may be opened, the upper door be held against its seat by the compressed air in the air-lock. As soon as door opens, the men and material may be passed into the shaft and chamber. In coming out the operations are reversed; men and mate the air-lock through the open lower door, the lower door is closed and he against its seat, and the equalizing valve is shifted, affording a conne tween the interior of the air-lock and the external air. The comp escapes through the equalizing valve, reducing the pressure in the a atmospheric pressure, and the upper door has atmospheric pressure sides of it. It may then be opened, giving free connection with the o

The Design of Pneumatic Caissons. The first consideration is, of have the final structure a permanent and sufficient pier to carry the l imposed upon it. To this end the cross-section of the pier at all points to bottom should be capable of carrying safely the maximum load cross-section of the pier is generally, in the finished pier, composed of crete, the cross-section will be determined by the allowable load on the For piers the cross-section will generally be square or circular; for caisson will generally be not less than 6 ft in width, as it is difficu caissons having a width less than 6 ft. If the caisson is to be carrie rock, the bearing on the rock need be no larger than the cross-secti concrete pier; but if the excavation does not go to rock, it is frequen able to BELL OUT the base of the pier so as to reduce the loading on th tion bed to a unit load less than that allowable on concrete. The op BELLING OUT is difficult in some materials; in a compact material it ca erally accomplished without serious difficulty.

Piers Sunk by the Pneumatic-Caisson Method may be constructed combinations of materials. The side walls and roof of the working were formerly frequently constructed of timber. In many cases they formed of steel; but in recent designs the working-chamber is general of reinforced concrete, the only structural steel used being an angle

ing. In building up the working-chamber, the INTERIOR FORMS are
as to support the concrete which makes the roof. These are sub-
moved. The exterior forms may constitute a permanent part of the
which case they are called a COFFER-DAM, or they may be removed
e concrete has sufficiently set. At the center of the pier an open-
to serve as the
pening connecting
chamber with the
The sides of this
of the upper part
are lined with an
TEEL SHELL. To
end of the steel
lock is connected.
t of the pier does
o ft the construc-
ay be completed
xcavation is com-
nerally, however,
tion of the pier is
oon as the work-
and from 5 to 10
uperimposed pier
onstructed; then
avation is done,
use of compressed
the cutting-edge
ter-level. This is
ING THE CAISSON
o that the caisson
me slight lateral
n the soil before
tion is carried up
to make it top-
en the entire pier
ection is finished,
resumed and the
ure is sunk as the
rogresses, care be-
to remove any
from beneath the

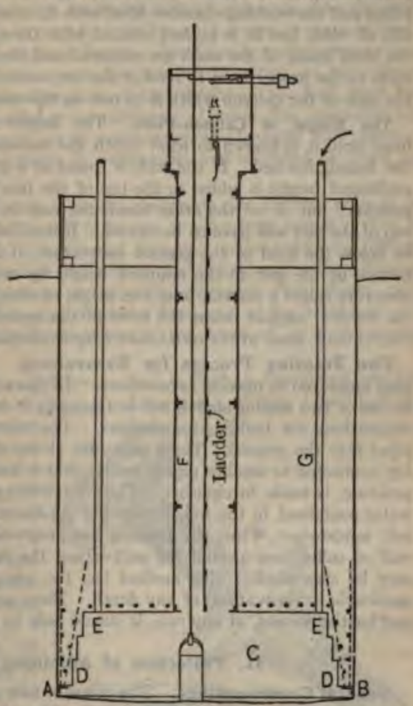

Fig. 45. Typical Form of Reinforced-concrete Caisson

During the progress of sinking compressed air is conducted to
chamber through the SUPPLY-PIPE *G*, the excavated material being
gh the SHAFT *F*. The shaft *F* is fitted with a LADDER for the use
en.

Caisson-Sinking and Filling. In sinking the caisson and super-
care must be taken to maintain it in a vertical position. This
accomplished in large caissons by means of the excavation itself.
ide of the caisson is high the excavation on that side will be carried
advance of the excavation on the low side, and the material under
dge of the high side will be removed while a bank of material is
he cutting-edge of the low side. These methods, however, are of
hen the caisson is narrow. In such cases that part of the caisson
ve ground is held in position by GUIDES or other devices; but it

The Height of Caisson-Piers. The height of a pier cannot
fixed until it is known to what depth the caisson must sink in or
the foundation-bed. If the rock is found at a greater depth than
additional height is added to the top of the pier after the caisson
position; but if, on the other hand, the rock is found unexpected
top of the pier will have to be cut off. If the finished elevation of
be below the level of the general excavation, it is usual to extend
surface of the pier to the required height by means of a tempora
structure called a COFFER-DAM, the height of which corresponds to
the finished surface below the level of the general excavation. In
COFFER-DAM some STEEL GRILLAGES may conveniently be set.

The Freezing Process for Excavations. This method has
been employed in making excavations. In this country its use has
to one or two mining-shafts, but in Germany it has been resorted
excavations for building-foundations. The method consists in
pipes into the ground. These pipes are closed at the bottom and
are connected to smaller pipes through which brine, at an extremel
perature, is made to circulate. The refrigerating effect results in
water contained in the soil, converting quicksand to a frozen mass
soft sandstone. When the freezing has progressed sufficiently to
wall or coffer-dam around the excavation, the material inside the
may be excavated. This method has the advantage, theoreticall
applicable to excavations of any depth. There are many precautions
and for the present, at any rate, it should only be considered as a pos

31. Protection of Adjoining Structures

General Considerations. The COMMON LAW provides that any
ing an excavation is responsible for resulting damage to adjoining
STATUTE LAWS as embodied in the building codes of different cities
or limit this responsibility, but in general, excavations should be m
a manner as to cause the least possible damage to surrounding propert
there are no adjoining structures it is generally sufficient to slope the
excavation so as to prevent the sliding of material into the excava

y means of heavy inclined posts called SHORES, arranged to act)LUMNS or STRUTS. Each SHORE consists of a post, the lower ests on a PLATFORM, generally consisting of planks and timbers to form a temporary spread footing. This platform should be pth which will insure that subsequent operations will not under- upper end of the post fits into a hole or niche cut into the wall d. The post itself may be a timber with a square cross-section, 12 in, and of the required length. Provision is made, between the the lower end of the post, for WEDGES or JACKS, so that when lifting effect transfers part of the weight of the wall from its temporary foundation or platform. During this operation all emporary structure are in compression and brought into bearing, ial under the platform is compressed and solidified as much as

ores. If the SHORE is to act preferably for LIFTING only, it is vertical as possible and is known as a LIFTING SHORE. If it is bly to combine a horizontal PUSHING action with the lifting ced at a considerable angle from the vertical and is then known SHORE or STEADYING SHORE. In arranging such shores care n to have the niche cut close to a floor-level of the building to be rwise the horizontal component of the thrust of the shores might ll.

d Sizes of Shores. Where a wall is light, a number of smaller be used in preference to a few large ones. Where a wall is high, shores of varying be used, and these tly be placed in the plane and rest on the same platform.

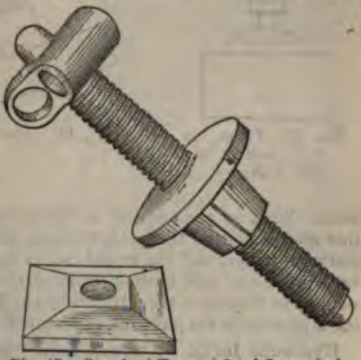

Fig. 47. Standard Type of Steel Screw-jack

Wedges and Screw-Jacks. In transferring the load of a wall from its own footing to the temporary platform, use is made of wooden rd or steel WEDGES, eel SCREW-JACKS, or HYDRAULIC dges and jacks may be used in combination. Wooden wedges de of hard wood and are generally arranged in pairs, both driven at the same time. The lifting effect of such wooden erful, but where a considerable settlement of the temporary anticipated, it is more convenient to use screw-jacks, as they considerable settlement.

nd Types of Screw-Jacks. The SCREW-JACKS usually manu- is purpose are made of cast iron and have rough threads, with ch to have much lifting effect. Screw-jacks of a better kind are and have a machine-thread of small pitch. Such jacks can be le of lifting weights up to 100 tons. Figs. 46 and 47 represent standard forms of screw-jacks. When a single screw-jack is used tion with a post or shore, a hole to receive the threaded portion is bored in the end of the timber used for the shore, the end being receive the nut. Such an arrangement is called a PUMP and is ill Fig. 48. When a lifting effect greater than that exerted by a single quired, the jacks are arranged in pairs in connection with a short timbe

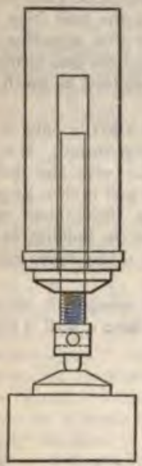

Fig. 48. Pump, or Screw-jack let into End of Shore

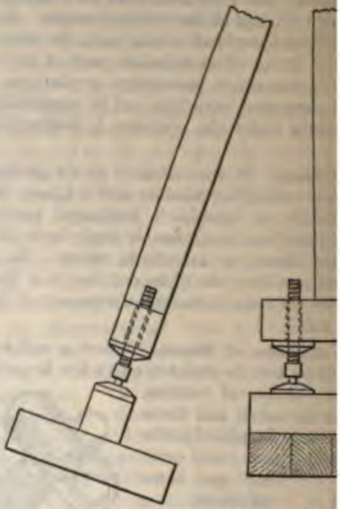

Fig. 49. Shore, Screw-jacks and Tim head

HEAD. Such an arrangement is illustrated in Fig. 49. It has the that after operating the jacks, blocking and wedges can be placed platform-timbers and the cross-head so that the post resting on the has a direct and solid bearing on the platform. By this method the wall can be thrown on the platform by the jacks and after the blockin ing is in position the jacks can be removed.

Hydraulic Jacks. Where excessively heavy loads are to be lifted, JACKS may be used in place of screw-jacks but an objection to them are liable to SLACK BACK under the load. While the load, there not be permanently supported on hydraulic jacks, they may be used load temporarily while the blocking and wedging are being placed cross-head and the temporary footing. In this way an indefinite shores may be set and taken care of with a single pair of hydraulic ja

Example of Shoring. Fig. 50 shows the method used in SHORIN mental front wall of a heavy building, advantage having been taken merous deep margin-drafts shown in the section. In order to avoid t of cutting niches for the tops of the shores, nine hardwood blocks were fitted to the margin-draft grooves in the masonry. Nine sim *b*, *b*, etc., were gained into and bolted to the vertical timber *VV*,

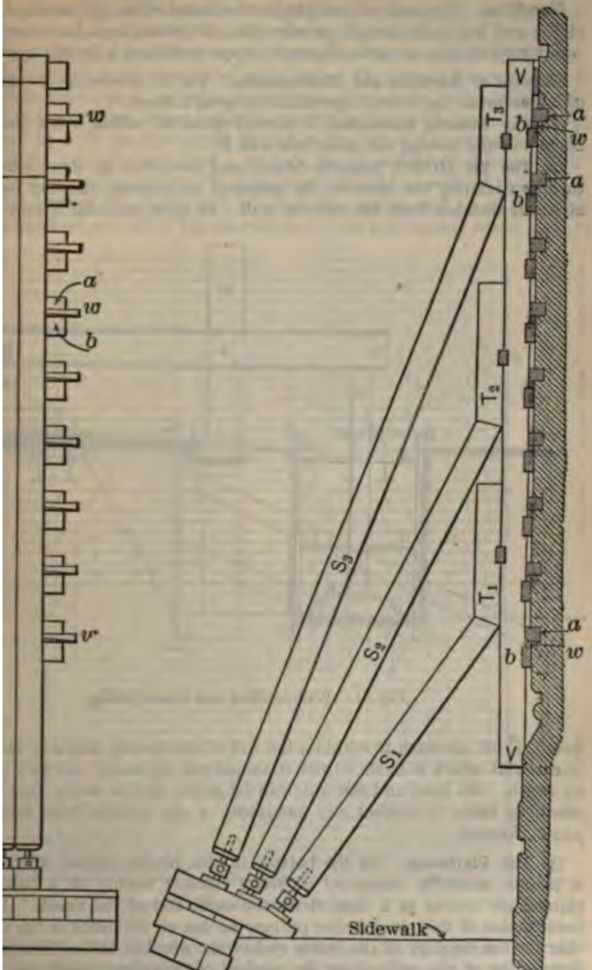

Fig. 50. Shoring an Ornamented Wall

n the *a* blocks and the *b* blocks for adjusting wedges *w*, *w*, etc. Three T_1, T_2 and T_3 were keyed and bolted to *VV* and transmitted to it f the three shores, S_1, S_2 and S_3. Each shore had a 60-ton screw-jack . Each shore is shown fitted with a pump or detached extension-ged for the screw-jack.

Needling. NEEDLES or GIRDERS are employed when part or all of t of the wall has to be carried, as when the old footing is to be remove wall UNDERPINNED or carried down to a new footing at a greater dept

Example of Needling and Underpinning. Fig. 51 represents a typ UNDERPINNING, the several operations being as follows:

(1) **The General Excavation** is carried down to within a few inc bottom of the footing *BB* under the wall *W*.

(2) **The Pit** *DDDD*, properly braced and protected by sheet pili to approximately the level of the proposed excavation, this PIT be at a safe distance from the existing wall. In good material it may

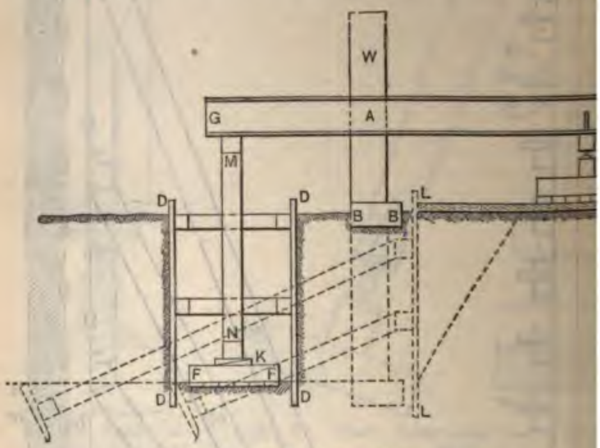

Fig. 51. Wall-needling and Underpinning

have this pit approach to within a few feet of the footing course of the in material which is liable to run it should not approach the wall c its depth. No hard and fast rule can be given, and in every case should be taken to prevent any movement of the material from und joining footing.

(3) **The Platforms.** On the bottom of this pit-excavation, a PLA is placed, generally composed of heavy timbers resting on a base planks, and acting as a support for the outer end of the needle. D construction of this pit a similar pit may be dug on the inside of the w vide for the support of the inside end of the needle; but as this in destruction of the cellar-floor the method of procedure inside the generally different from this. If the material is solid it is sometimes to place the platform for the support of the inside end of the need on the cellar-floor and at such a distance from the wall that the excavation for the new footing will not disturb it; or the platform placed on the cellar-floor and a line of sheeting *LL*, properly braced, that the excavation can be made for the new footing. This is generally to prevent any serious settlement of the temporary platform for the of the needle.

ion of the Needles. Having provided a support for each end
only remains to cut a hole through the wall, as at *A*, insert the
he post and blocking *MN* under the outside end of the needle,
and jacks under the inside end. The post *MN* may be fitted
hown at *K*, or with one or more screw-jacks. The needle *GG*
e or more heavy timbers or one or more steel **I** beams. In any
come on this needle should be figured and its strength made
support such load. As soon as the weight of the wall *W* is
e needles and to the temporary platforms prepared to receive
rt of the wall which is below the needles and all of the foot-
ved and all of the excavation for the new footing made.

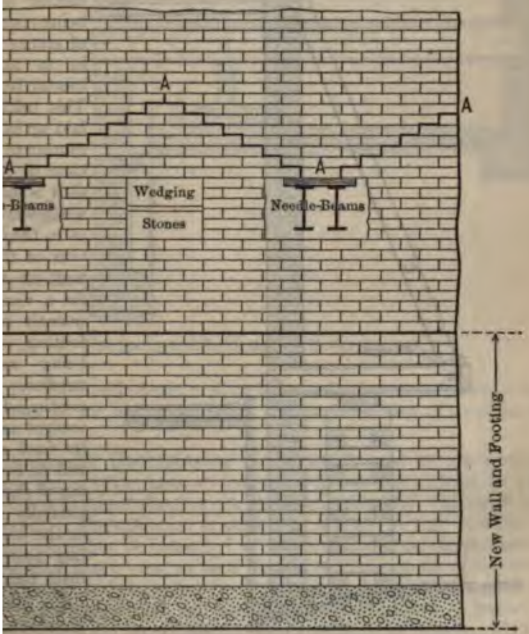

Fig. 52. Needling a Brick Wall

Brick Wall. Fig. 52 shows the elevation of a brick wall sup-
ES. If the needles are carrying the entire weight of the wall, it
at the level of their upper surfaces the entire weight will be
gh those parts of the wall which are immediately above them,
these points the material composing the wall will CORBEL OUT
s as indicated in Fig. 52 by the heavy zigzag lines *AAAAA*.
low this line will be supported simply by COHESION to the part
it. An experienced man can determine the location of this
d given by the wall on being struck by a hammer. All of the
ne is HANGING and liable to fall as soon as the support given by

the footing is removed. The hanging parts of the wall may be remov pended by rods and chains to the needles. If they are not so su crack will form along the line *AAAAA*.

Transferring the Load to the New Underpinning. As soon a footing has been put in place and the new wall carried up ready to r old wall, provision must be made for REVERSING THE OPERATION, t transferring the load onto the new underpinning wall and footing generally done by means of a number of GRANITE BLOCKS set in pai the needles and fitted with STEEL WEDGES. After setting these blocks between the base of the old wall and the top of the upper wedging bl in with brick mortar in the being compacted of PIECES OF SL in so as to mortar between This brickwork laid up in Portla mortar so as to time of setting as it is sufficient wedges are driv as to throw at l tion of the wei wall on the ne As a result of quently happen footing settles, being restored needles. This n continued drivi wedges until it h its final settlem will be evidence ing of the wall st partially relieve in the needles a fact that the v main tight.

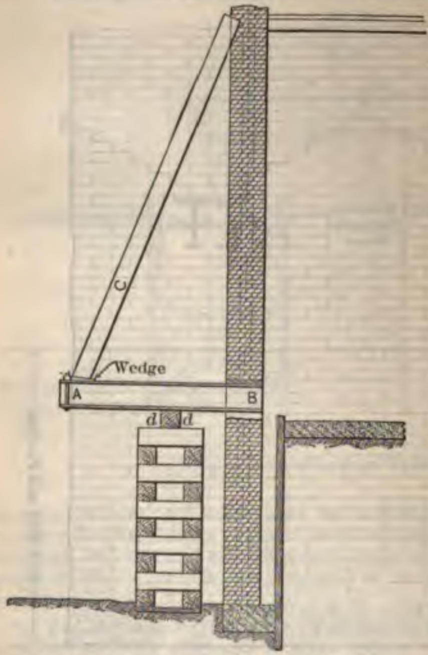

Fig. 53. The Figure-four Method of Needling

Removal Needles, etc. as the entire we wall has been t to the footing and the footing has demonstrated that it is capab porting the weight of the wall without further settlement, all of the work, including the needles, can be removed, the needle-holes brick the repairs made to the cellar of the adjoining building.

The Figure-Four Method of Needling. In certain cases it is i to employ a NEEDLE-BEAM projecting on both sides of the wall, as fo when the occupancy of the adjoining building is such as to make it i to have a needle-beam projecting into the cellar space. In such ca called FIGURE-FOUR NEEDLE has been employed (Fig. 53). In thi needle *AB* acts as a CANTILEVER. Part of the load of the wall is carr ore *C* and another equal or nearly equal part is carried by the needle eedle-beam *AB* being really balanced on the block *dd*.

-Needles. Fig. 54 shows a method frequently employed, known as -NEEDLE METHOD. In this case the needle engages with the wall to ted and also with an adjoining wall. A temporary platform is placed o the wall to be supported, *W*, as is practicable. The uplift of the ling to lift the needle-beam acts on both walls, but on account of its ated nearer to the wall to be lifted, a large proportion of its effect is hereon.

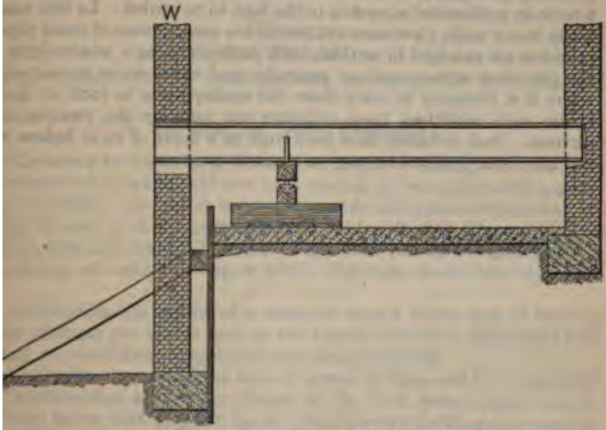

Fig. 54. The Spring-needle Method of Underpinning

or Cylinders for Underpinning are frequently used for the support and have many advantages, as they not only afford a support for the hrough the operations affecting the stability of the wall, but also form ent support. The operation in brief is as follows: A hole or niche is wall and footing to be supported, of sufficient size to permit the intro-of a section of STEEL PIPE, in such manner that the center of the pipe below the center of the wall to be supported, the height being sufficient mmodate a section of pipe and also the means employed to drive it. may be driven (1) by HYDRAULIC JACKS or by SCREW-JACKS, placed the top of the pipe and the wall itself, as by the patented BREUCHAUD (2) it may be driven by means of a POWER-HAMMER driven either by compressed air; or (3) in some cases, where the material is fine sand or pipe may be JETTED or the JET-METHOD may be used in combination er jacks or power-hammers. In any case the first section of pipe is to the ground and additional sections are added until the lower end of encounters rock or some material possessing sufficient stability to insure red support. The material entering the pipe is removed by a WATER- other means and the space filled with concrete. As soon as the con-set sufficiently the pipe is capped with a special casting on which short ams are arranged to distribute the support of the pipe over a consider-of the base of the wall to be supported. These **I** beams correspond to the wedging blocks used in the ordinary methods before descri vision is frequently made for STEEL WEDGES between the cap and the steel beam, but it is generally found sufficient to thoroughly grout in between the base of the wall and the steel beams after the niche its bricked up.

Cylinders for Underpinning Very Heavy Walls. The descrip preceding paragraph is intended to cover the use of pipes varying in 6 to 20-in in diameter, according to the load to be carried. In the ca sively heavy walls, CAST-IRON CYLINDERS are used in place of steel pi cylinders are arranged in sections, each section making a water-tight the preceding section, and are generally used where water is encou where it is necessary to carry down the underpinning to rock at gr Under such conditions these cylinders are sunk by the PNEUMA METHOD. Such cylinders have been sunk to a depth of 70 ft below and have been designed to carry as much as 400 tons.

1. Footings for Light Buildings

ourses in General.† Every foundation or bearing wall overlying
cept solid rock should rest on a footing or base projecting beyond
ach side. On wet or very compressible soils these footings may be
l beams or of reinforced concrete, as described in Chapter II, but
y firm soils and for buildings of moderate size and weight the foot-
erally of concrete, stone, or brick. Footings answer two important

stributing the weight of a structure over a larger area of bearing
pressure per square foot on the foundation-bed is diminished and
y to vertical settlement correspondingly lessened.
creasing the area of the base of a wall, footings add to its stability
protection against the danger of the work being thrown out of
ny forces that may act on it. Nearly every building law requires
foundation wall or pier and every cellar or basement wall or pier
footing at least 12 in wider, that is, 6 in on each side, than the thick-
vall or pier, and this may be considered as the minimum projection,
re instances where there may be a special reason for making it less.
s and for comparatively light buildings a projection of 6 in on each
ll will generally reduce the unit pressure, that is, the pressure per
to the safe resistance of the soil, but it is always wise to proportion
to a uniform unit pressure, as explained in Chapter II, Subdivision
any useful effect, footings must be well bedded and have sufficient
trength to resist the upward reactions on the projections.

otings for Walls with Ordinary Loads. Stone cellar walls and
alls generally have stone footings, although if the walls are heavily
tom footing of coarse concrete is advisable under the stone footing.
le, stone footings should consist of stones having a width equal to
footing. If impracticable to obtain stones of this size, then two
d be used, meeting under the middle line of the wall. In any event
course should extend inside of the course above, a distance equal to

Stone footings should be of hard, strong and durable stones, alway their natural bed and solidly bedded in mortar. As a general rule, buildings, and where the loads per unit of foundation-bed are much the allowable pressure, the thickne course is made about equal to its p beyond the course above. The m mon defect in large-stone footing the stones are not properly bedded more difficult to bed a large ston small one. The stones should be thick bed of mortar and worked side a bar until firmly settled in place.

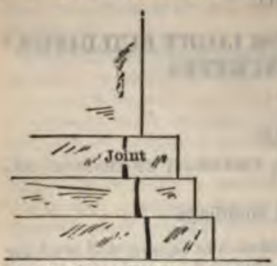

Fig. 1. Stone Footing. Openings at Joints

Offsets in Masonry Footing projections of the footing course the wall, or beyond the courses ab be carefully considered, whatever terial of the footings. If the pro the footing or offset of the cours great for the strength of the stone, brick, or concrete, the footing as shown in Fig. 2. The proper offset for each course depends vertical load, the transverse strength of the material, the resisting power of the foundation-bed and the thickness of the course.†

Fig. 2. Crack from Excessiv

Tables for Offsets for Masonry Footing Courses.‡ As stated in Chapter II, in the discussion of the design of stepped footings, there are rule-of-thumb methods giving so-called safe projections for given depths of footings or giving the ratios between the projections and the depths of the courses. Tables of offsets for footing courses of different materials have been computed from the flexure-formula applied to the projecting footing courses considered as CANTILEVER BEAMS uniformly loaded by the upward pressures on the u Although these tables, so computed, are incorporated in some build they cannot be safely used without numerous restrictions, exceptions fications, and hence they are, in general, unreliable and of use only a mations. As these tables are still inserted in engineers' and archite books, the table of offsets for masonry footing courses, in a revised f tained in this chapter with the recommendation that for footings offsets it be used with caution and that for such footings the methods in Chapter II be used when greater accuracy of results is required.

Notes Regarding Use of Table I. The values in Table I are from the formula $l = \frac{1}{6} d \sqrt{S_f / wf}$ which is derived from the FLEXURE for a uniformly loaded cantilever beam, and slightly changed to numerical coefficient of the second member of the equation the valu In this equation, $l =$ the maximum allowed offset of the footing course $d =$ the thickness of the footing course in inches, $S_f =$ the modulus

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* See Offset Footings, Chapter II, especially Subdivisions 17 and 22.

† See Chapter II, Subdivision 22, for a complete discussion of the principl in the design of projecting footings, ratio of projection to depth of footing, etc. geneous slabs, separate-layer footings, etc.

‡ See Chapter II, Subdivision 22, page 180.

§ See, also, formula in Chapter II, Subdivision 22.

erage)	[illegible]	[illegible]	[illegible]	[illegible]
(average)	1 375	2.8	2.0	1.4
(average)	1 375	2.8	2.0	1.4
(good bricks in natural-cement mortar, lays old)	125	0.8	0.6	0.4
(hard-burned bricks in Portland-cement 1 : 3, 60 days old)	400	1.6	1.1	0.7
Portland cement, 1 : 2 : 4, 1 month old)	300	1.3	0.9	0.6
Portland cement, 1 : 2 : 4, 6 months old)	400	1.5	1.1	0.7

erials in pounds per square inch, w = the determined or assumed the bottom surface of the footing course considered, in tons of 2 000 , and f = the factor of safety used. The table gives the values of l/d nit pressures w. For example, if w is taken at 2 tons per sq ft, then ne or sandstone footings $l/d = 1.4$, and if d, the thickness of the ırse, is 12 in, the offset or projection should be 16 or 17 in. The n in the table for S_f, the modulus of rupture for the materials, differ ly from those given in Subdivision 22 of Chapter II, in Table I of V and in Table III of Chapter XVI. If results are required based ent fiber-stresses, upon a different factor of safety, or upon different er square foot, the formula may be used instead of the table. It ıys be borne in mind that as each footing course transmits the entire he wall and its load, the pressure will be greater per square foot on courses, and that the offsets should be made proportionately less; he values in Table I, when applied to stone-masonry footings, are l for the lower offset only, and then only when the footing is built he thickness of which is equal to the thickness of the course, which jection of less than half their length, and which are well bedded in rtar.

e Footings.* For buildings of great weight, except the very heaviest, lly for those built on a clay soil, concrete generally makes the best d it is even preferable to and generally cheaper than large slabs of ıen the concrete is properly made and used, it attains a strength concrete. The proportions and manner of mixing concrete are de latter part of this chapter.

Depositing the Concrete. Concrete should be used as soon a should always be deposited in layers, which as a rule should not e thickness, especially for the first layer. On small jobs where the by hand the concrete is usually carried to the trenches in wheel- dumped into the trenches. The height from which the concret however, should not exceed 4 ft above the bottom of the trench, be falls from a greater height the heavy particles are apt to separ lighter ones. As soon as the concrete has been deposited in the should be leveled off and then tamped with a wooden rammer we 20 lb, until the water in the concrete is brought to the surface. Co not be permitted to dry too quickly, and if twenty-four hours el the deposits of the successive layers, the top of each layer should before the next is put in place. For buildings over five stories high idea to place a stone footing course above the concrete footing, if su for the purpose can be obtained.

Brick Footings. Where the foundation walls are of brick, the usually brick or concrete. For interior walls on dry ground, a

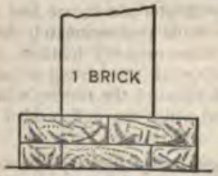

Fig. 3. Brick Footing. Wall One Brick Thick

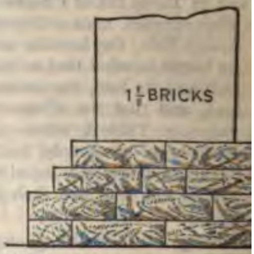

Fig. 4. Brick Footing. Wall One-half Bricks Thick

m one brick to three bricks in thickness. The bricks used for foot- be the hardest and soundest that can be obtained, should be laid in rtar and should be either grouted or thoroughly slushed up, so that shall be entirely filled with mortar. The writer favors GROUTING footings, that is, the thin mortar to fill the ts, as he has always it gives very satisfac- s. The bottom course ting should always be bed of mortar spread ottom of the trench latter has been care- ed. All bricks laid in lry weather should be wet before laying, for, , they rob the mortar percentage of the mois- tains, greatly weakening the adhesion and strength of the mortar. tention should be given to the laying of the footing courses of as upon them the stability of the work largely depends. If the urses are not solidly bedded, if any rents or voids are left in the e masonry, or if the materials themselves are unsound or badly er, defects in the superstructure are almost sure to show them- er or later, and almost always at a period when remedial efforts are d expensive.

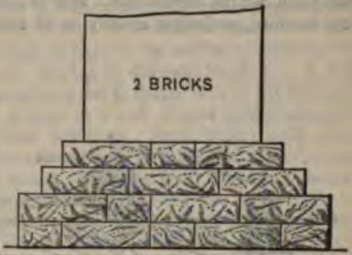

Fig. 5. Brick Footing. Wall Two Bricks Thick

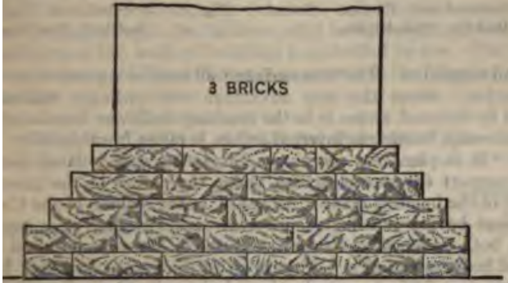

Fig. 6. Brick Footing. Wall Three Bricks Thick

l Arches.* In a few buildings in which the external walls are o piers with wide openings between them, and in which the support- f the soil is not more than 2 or 3 tons per sq ft, it was thought desir- nect the bases of the piers by means of INVERTED ARCHES, for the pur- ributing the weight of the piers over the whole length of the footings. f inverted-arch footings are shown in Figs. 7 † and 8,† which represent y the construction employed in the Drexel Building in Philadelphia

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example worked out in full, showing the method of proportioning inverted Chapter III, Building Construction and Superintendence, Part I, Masons' . E. Kidder.

e Engineering Record, May, 1899, and Nov., 1890.

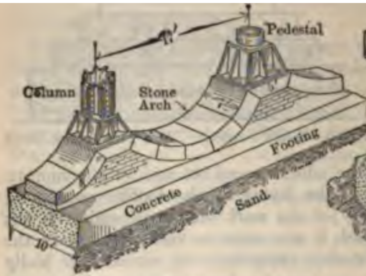

Fig. 7. Inverted-arch Footing. Drexel Building, Philadelphia

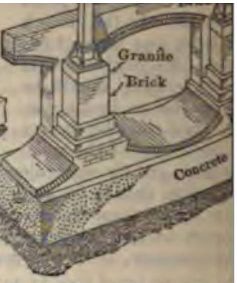

Fig. 8. Inverted-arch Footi Building, New York

to the load supported. The intermediate wall may be supported by st or by arches. About the only advantage over ordinary masonry possessed by inverted arches is in the resulting shallower foundations

The following, relating to inverted arches, is taken from the New Y ing law: "If, in place of a continuous foundation wall, isolated piers built to support the superstructure, where the nature of the groun character of the building make it necessary, in the opinion of the Con of Buildings having jurisdiction, inverted arches resting on a prop concrete, both designed to transmit with safety the superimposed lo be turned between the piers. The thrust of the outer piers shall be by suitable wrought-iron or steel rods and plates." (Law of 1906.)

2. Cellar Walls and Basement Walls

Definitions. These terms are generally applied to walls which the surface of the ground or below the water-table or first-flo which support the superstructure and which go down to the FOUNDATI properly so called. (See Chapter II, Divisions 1 and 20.) Walls w thick, and should be well bonded, with full and three-quarter paces between the stones should be filled solid with mortar s or spalls. The MORTAR for stonework should be made of p and rather coarse sand. The outside walls of cellars ould be PLASTERED smooth on the outside with 1 : 2, or 1 : 1½ om ½ to ¾ in thick. In heavy-clay soils it is a good idea to on the outside, making them from 6 in to 1 ft thicker at the ne top.

Cellar and Basement Walls. This is usually governed by bove, and also by the depth of the wall. Nearly all building that the thickness of the cellar and basement wall, to the w the grade-line, shall be 4 in greater than the thickness of the ck, and 8 in greater for stone, and that for every additional of in depth, the thickness shall be increased 4 in. In all large of the walls of buildings is controlled by law. For buildings ness of the walls is not so governed, the following table will

II. Thickness of Cellar and Basement Walls

f building	Dwellings, hotels, etc.		Warehouses	
	Brick, in	Stone, in	Brick, in	Stone, in
	12 or 16	20	16	20
	16	20	20	24
	20	24	24	28
	24	28	24	28
	28	32	28	32

tects when designing brick buildings. Table III * gives the thicknes
walls required for MERCANTILE BUILDINGS in representative cities of
sections of the United States, and affords about as good a guide a
have, because the values given, as a rule, represent the judgment
qualified and experienced persons. Walls for DWELLINGS are gene
mitted to be 4 in less in thickness than for warehouses, although
cities little or no distinction is made between business blocks and dw

Table IV gives the thickness required for the brick walls of dwelli
ments, hotels and office-buildings in Chicago. The thickness given is
mum that should be allowed for the walls of such buildings. Most citi
13-in walls for the upper story of three-story buildings and for l
story dwellings. In St. Louis the two upper stories of dwellings are t
be 13 in, the next two below, 18 in, the next two 22 in, and the next
thick.

In compiling Table III the top of the second floor was taken at 1
the sidewalk, and the height of the other stories at 13 ft 4 in, incl
thickness of the floor, as the New York and Boston laws and the la
other cities give the height of the walls in feet instead of in stories.
height of stories exceeds these measurements the thickness of the wal
cases will have to be increased. The Chicago ordinance (1914) spe
"where 12-in walls are used, the story-heights shall not exceed 18 ft, w
walls are used, the story-heights shall not exceed 24 ft, and where 2
are used, the story-heights shall not exceed 30 ft."

General Rule for Thickness of Walls. Although there are gr
ences in the thicknesses given in Table III, more indeed than there
a general rule might be formulated from it, for MERCANTILE BUILD
four stories in height, which would be somewhat as follows:

For bricks equal to those used in Boston or Chicago, make the th
the three upper stories 16 in, of the next three below 20 in, the next t
and the next three 28 in. For a poorer quality of material make onl
upper stories 16 in thick, the next three 20 in, and so on down. In bui
than five stories in height the top story may be 12 in thick.

In determining the thickness of walls the following general principl le III.* Thickness in Inches of Walls for Mercantile Buildings and, Except in Chicago, for All Buildings Over Five Stories in Height

Height and location of building		Stories							
		1st	2d	3d	4th	5th	6th	7th	8th
o es	Boston	16	12	....	....	....	....	....	....
	New York	12	12	....	....	....	....	....	....
	Chicago	12	12	....	....	....	....	....	....
	Minneapolis	12	12	....	....	....	....	....	....
	St. Louis	18	13	....	....	....	....	....	....
	Denver	13	13	....	....	....	....	....	....
	San Francisco	17	13	....	....	....	....	....	....
	New Orleans	13	13	....	....	....	....	....	....
ee es	Boston	20	16	16	....	....	....	....	....
	New York	16	16	12	....	....	....	....	....
	Chicago	16	12	12	....	....	....	....	....
	Minneapolis	16	12	12	....	....	....	....	....
	St. Louis	18	18	13	....	....	....	....	....
	Denver	17	17	13	....	....	....	....	....
	San Francisco	17	17	13	....	....	....	....	....
	New Orleans	13	13	13	....	....	....	....	....
r es	Boston	20	16	16	16	....	....	....	....
	New York	16	16	16	12	....	....	....	....
	Chicago	20	16	16	12	....	....	....	....
	Minneapolis	16	16	12	12	....	....	....	....
	St. Louis	22	18	18	13	....	....	....	....
	Denver	21	17	17	13	....	....	....	....
	San Francisco	17	17	17	13	....	....	....	....
	New Orleans	18	18	13	13	....	....	....	....
e es	Boston	20	20	20	20	16	....	....	....
	New York	20	16	16	16	16	....	....	....
	Chicago	20	20	16	16	16	....	....	....
	Minneapolis	20	16	16	12	12	....	....	....
	St. Louis	22	22	18	18	13	....	....	....
	Denver	21	21	17	17	13	....	....	....
	San Francisco	21	17	17	17	13	....	....	....
	New Orleans	18	18	18	13	13	....	....	....
es	Boston	24	20	20	20	20	16	....	....
	New York	24	20	20	20	16	16	....	....
	Chicago	20	20	20	16	16	16	....	....
	Minneapolis	20	20	16	16	16	12	....	....
	St. Louis	26	22	22	18	18	13	....	....
	Denver	26	21	21	17	17	13	....	....
	San Francisco	21	21	17	17	17	13	....	....
	New Orleans	22	18	18	18	13	13	....	....
n es	Boston	24	20	20	20	20	20	16	....
	New York	28	24	24	20	20	16	16	....
	Chicago	20	20	20	20	16	16	16	....
	Minneapolis	20	20	20	16	16	16	12	....
	St. Louis	26	26	22	22	18	18	13	....
	Denver	26	21	21	21	17	17	17	....
	New Orleans	22	22	18	18	18	13	13	....
t es	Boston	28	24	20	20	20	20	20	16
	New York	32	28	24	24	20	20	16	16
	Chicago	24	24	20	20	20	16	16	16
	Minneapolis	24	20	20	20	16	16	16	12
	St. Louis	30	26	26	22	22	18	18	13
	Denver	30	26	21	21	21	17	17	17
	New Orleans	22	22	22	18	18	18	13	13

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* See footnote on page 230.

Table III (Continued).* Thickness in Inches of Walls for Merca Buildings and, Except in Chicago, for all Buildings Over Five Stories in Height

Height and location of building		Stories									
		1st	2d	3d	4th	5th	6th	7th	8th	9th	10t
Nine stories	Boston	28	24	24	20	20	20	20	20	16	...
	New York	32	32	28	24	24	20	20	16	16	...
	Chicago	24	24	24	20	20	20	16	16	16	...
	Minneapolis	24	24	20	20	20	16	16	16	12	...
	St. Louis	30	30	26	26	22	22	18	18	13	...
	Denver	30	26	26	21	21	21	17	17	17	...
Ten stories	Boston	28	28	24	24	20	20	20	20	20	16
	New York	36	32	32	28	24	24	20	20	16	16
	Chicago	28	28	24	24	24	20	20	20	16	16
	Minneapolis	24	24	24	20	20	20	16	16	16	12
	St. Louis	34	30	30	26	26	22	22	18	18	13
	Denver	30	30	26	26	21	21	21	17	17	17
Eleven stories	Boston	36	32	32	28	28	24	20	20	20	[illegible]
	New York	36	36	32	28	28	24	24	20	20	16
	Chicago	28	28	24	24	24	20	20	20	16	16
	St. Louis	34	34	30	30	26	26	22	22	18	18
	Denver	30	30	26	26	26	21	21	21	17	17
Twelve stories	Boston	36	36	32	32	28	28	24	20	20	[illegible]
	New York	40	36	36	32	32	28	24	24	20	[illegible]
	Chicago	28	28	28	24	24	24	20	20	20	16
	St. Louis	34	34	34	30	30	26	26	22	22	18
	Denver	30	30	30	26	26	26	21	21	21	17

* See footnote on page 230.

Table IV.† Thickness of Enclosing Walls for Residences, Tenen Hotels and Office-Buildings. Chicago Building Ordinance

Number of stories	Basement	Stories									
		1st	2d	3d	4th	5th	6th	7th	8th	9th	10t
Basement and One-story	12	8	....	....	....	....	....	....	....	....	....
Two-story	12	12	8	....	....	....	....	....	....	....	....
Three-story	16	12	12	8	....	....	....	....	....	....	....
Four-story	20	16	16	12	12	....	....	....	....	....	....
Five-story	20	16	16	16	12	12	....	....	....	....	....
Six-story	20	20	16	16	16	12	12	....	....	....	....
Seven-story	24	24	20	20	16	16	12	12	....	....	....
Eight-story	24	24	24	20	20	16	16	12	12	....	....
Nine-story	28	24	24	20	20	20	16	16	12	12	....
Ten-story	28	24	24	24	20	20	20	16	16	12	12
Eleven-story	28	28	24	24	24	20	20	20	16	16	12
Twelve-story	32	28	28	24	24	24	20	20	20	16	16

† These thicknesses are allowed when certain requirements are fulfilled in lengths of walls, heights of stories, etc. For these modifying restrictions a classifications of buildings in regard to their uses the building laws must b The table is inserted in this form as a useful general guide and as an illust average contemporary practice.

h stories and clear spans exceeding 25 ft require thicker walls.
length of a wall is a source of weakness, and that the thickness
eased 4 in for every 25 ft over 100 or 125 ft in length. In New
nesses given in the table must be increased for buildings exceed-
lepth on the lot. In Western cities the tables are compiled fo
25 ft in depth, as that is the usual depth of lots in those

lls containing over 33% of openings should be increased in thick-

rtition walls may be 4 in less in thickness than the outside walls
ft long, but that no partition should be less than 8 in thick.

ed with Ashlar. "In reckoning the thickness of walls, ASHLAR
cluded unless the walls are at least 16 in thick and the ashlar is
hick, or unless alternate courses are at least 4 and 8 in to allow
the backing. Ashlar shall be properly held by metal clamps to
properly bonded to the same." *

lls should generally be 4 in thicker than required for brick

alls.† Hollow walls are undoubtedly desirable for dwellings, and
ised for other buildings not more than four or five stories in height,
the security afforded from the weather. Owing to the fact that
lly more expensive than solid walls and occupy more space, they
xtensively used in this country.
building law requires that "vaulted walls shall contain, exclusive
same amount of material as is required for solid walls, and the
e inside of the air-space in walls over two stories in height shall
an 8 in thick, and the parts on either side shall be securely tied
ties not more than 2 ft apart in each direction."

oncrete Blocks. Blocks made of Portland-cement concrete, and
ds, are frequently used for building walls and partitions that are
thin and bear light loads. Patents have been taken out on
s of blocks and on machines or processes for making the same,
dings have been erected with walls built of these blocks. Most of
molded so as to form hollow walls. Block construction of this
dvantage over poured walls, in that the blocks are thoroughly
e they are set and hence no provision is required for expansion or
For the thin, light walls above mentioned the concrete-block
s better adapted than solid concrete. The expense of forms is
lso the tendency to crack and to leave an unsatisfactory surface-
ete blocks may be substituted for any ordinary stone or brick
ilding laws usually require the thickness of walls of hollow con-
be not less than that required for brick walls. They should not
ty walls. (See, also, Chapter XXIII, Subdivision 2.)

Hollow Tiles. Hollow tiles are used for the external walls of
sometimes for factories in some locations and under certain
For example, the building laws (1913) of the District of Columbia
l hollow tiles, not less than 12 in in thickness, to be used for the

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lding Law, in force in 1915.
escription of the *details of construction of hollow, brick walls*, see pages
lding Construction and Superintendence, Part I, Masons' Work, by F. E.

because when heated and then suddenly cooled by water they are
from the sudden contraction. Recent conflagrations have show
burned terra-cotta will crack and fall to pieces under severe heat
also, Chapter XXIII, Subdivision 2.)

Party Walls. There is much diversity in building regulatio
the thickness of party walls, although they all agree in that such
never be less than 12 in thick. About one-half of the laws requi
walls shall be of the same thickness as external walls; the remain
equally divided between making the party walls 4 in thicker or
for independent side walls. When the walls are proportioned
previously given the author believes that the thickness of the party
be increased 4 in in each story. The floor-load on party walls
twice that on side walls, and the necessity for thorough fire-
greater in the case of party walls than in other walls.

Enclosing Walls for Steel, Skeleton Construction. In bui
skeleton type the outer masonry walls are usually supported eit
story or every other story by the steel framework, and carry noth
own weight. Such walls may, therefore, be considered as only
stories high, and are usually made only 12 in thick for the whole
twelve-story or fifteen-story building. For SKELETON CONSTR
Chicago ordinance allows ENCLOSING WALLS of 12-in thickness fo
The New York City building law requires the use of 12-in enclosing
ft of the uppermost height thereof, or to the nearest tier of be
measurement, and 4 in additional thickness for every lower 60-ft
the nearest tier of beams to such vertical measurement, down to
beams nearest to the curb-level. But, on account of the severity
the requirements as applied to very high buildings of skeleton c
permission is frequently given by the Commissioners of Buildin
empowered to modify the building laws within certain limits,
the thicknesses of curtain walls for very high buildings, a
the peculiar circumstances of each case, without endangering the s
safety of the building. A few of the earlier tall buildings were buil
SUSTAINING WALLS, starting from the foundation, while columns

ous limestone containing sufficient silica, alumina and iron oxide to
aulic properties when the burned rock was pulverized and gauged
These natural cements were very widely manufactured and used
t years, when they have been practically completely replaced by
ement. Natural cements vary in color from light yellow to dark
rding to the content of oxide of iron, and in distinction to Portland
ey are not uniform in their composition or behavior. The chemical
and physical characteristics of various natural cements vary within
not only between cements manufactured in different mills, but be-
products of the same mill at different times. Natural cements set
ly than Portland cements and are slower in developing strength.
tion of natural cement in the United States for 1913 was 800 000
ile during the same year the production of Portland cement was
barrels; from which it is seen that the natural-cement industry is
lmost extinct. Natural cement may be used in massive masonry
ht rather than strength is the essential feature. It is used, also,
special purposes, such as in the manufacture of safes and in certain
vhere a quick-setting cement is necessary. Where economy is the
actor, a comparison may be made between the use of natural cement
r mixture of Portland cement that will develop the same strength.

The specifications of the American Society for Testing Materials
a bag of natural cement shall contain 94 lb, net, of cement, and that
of natural cement shall contain three bags of this NET WEIGHT.

. A natural-cement mortar, in order to comply with the require-
e standard specifications of the American Society for Testing Ma-
t show a TENSILE STRENGTH, for the neat cement, of at least 150 lb
hen one week old, and 250 lb at the end of 28 days; or, when mixed
arts of standard Ottawa sand, 50 lb at the end of one week, and 125
d of 28 days. The strength of 1 : 2 natural-cement mortar is about
t of 1 : 4 Portland-cement mortar.

ons of Natural Cement and Sand for Mortar and Concrete.
for rubble-stone masonwork and ordinary brickwork, one part of
ent may be mixed with three parts of sand, by measure.

ic Lime. A product closely related to natural cement is HYDRAULIC
s is manufactured in the same way as natural cement, but the rock
ns sufficient lime to permit it to slake like quicklime. When the
roduct is pulverized, it sets and hardens as an hydraulic cement.
limes are largely manufactured in Europe, and especially in France
n, but in the United States they have been manufactured only in a
s. This is due to the fact that while rock of suitable composition
und, the impurities are not uniformly distributed through it, but are
yers or seams which prevent the material from being uniformly
he portion of the rock immediately adjacent to and including the
purities overburns, frequently melting like a slag, while the purer
nsist simply of quicklime; and while the resulting mass slakes partly,
when pulverized is unreliable as a cement.

l data relating to *Cements, Limes and Plasters were furnished the Editor by*
Warner Company of Wilmington, Del. For Limes and Plasters, see Part
to 1472.

The Artificial Cements used in the United States include Portl
and Puzzolan or slag cement.

Portland Cement. The principal artificial cement in this cou
is PORTLAND CEMENT. It is manufactured from two raw material
ground to extreme fineness to secure an intimate mix before burn
is from this fact that it derives its name, ARTIFICIAL CEMENT.
terials must be so proportioned that in the finished cement, silica, al
oxide and lime will be present in a certain ratio which must be
within close limits. In the Lehigh Valley region of Pennsylvani
are located some of the leading Portland-cement mills of the United
raw materials used are limestone and cement-rock. The cement-roc
pure limestone carrying argillaceous or clay-matter. In order to brin
content up to the required percentage, it is usually found necess
region to add limestone. In other districts the raw materials use
stone and clay, limestone and shale, marl and clay and also blast-f
and limestone. The product from the last-mentioned mixture sho
confused with the common slag cement or Puzzolan cement, as the sla
used as a raw material supplying silica, alumina, iron oxide and lime
the exception of the use of slag to furnish these ingredients, the proces
facture and the properties are substantially the same as for the othe
cements. The raw mix in a Portland cement mill is analyzed at
several times each hour to keep the composition of the cement within
limits. The raw material, which is pulverized as fine as the finished
burned in rotary kilns, the fuel used in most instances being powd
From the kiln it issues in the form of CLINKER, the name given to th
rified product. After cooling, calcium sulphate in the form of
added to control the set and the product is pulverized and packed for
The manufacture and properties of Portland cement have been made t
of careful study by the American Society for Testing Materials an

RD SPECIFICATIONS FOR PORTLAND CEMENT.* The following extracts
ost important requirements for Portland cement:

ION. This term is applied to the finely pulverized product resulting
alcination to incipient fusion of an intimate mixture of properly pro-
argillaceous and calcareous materials, and to which no addition
an 3% has been made subsequent to calcination.

GRAVITY. The specific gravity of cement shall be not less than 3.10.
e test of cement as received fall below this requirement, a second test
ade upon a sample ignited at low red heat. The loss in weight of the
nent shall not exceed 4%.

ss. It shall leave by weight a residue of not more than 8% on the
nd not more than 25% on the No. 200 sieve.

F SETTING. It shall not develop initial set in less than 30 minutes,
develop hard set in not less than 1 hour, nor more than 10 hours.

STRENGTH. The minimum requirements for tensile strength for
1 sq in in cross-section shall be as follows, and the cement shall show
ession in strength within the periods specified:

NEAT CEMENT

	STRENGTH
urs in moist air	175 lb per sq in
ys (1 day in moist air, 6 days in water)	500 lb per sq in
ys (1 day in moist air, 27 days in water)	600 lb per sq in

NE PART CEMENT, THREE PARTS STANDARD OTTAWA SAND

ys (1 day in moist air, 6 days in water)	200 lb per sq in
ys (1 day in moist air, 27 days in water)	275 lb per sq in

NCY OF VOLUME. Pats of neat cement about 3 in in diameter, ½ in
ss at the center, and tapering to a thin edge, shall be kept in moist
eriod of 24 hours.

at is then kept in air at normal temperature and observed at intervals
t 28 days.

ther pat is kept in water maintained as near 70° F. as practicable, and
t intervals for at least 28 days.

hird pat is exposed in any convenient way in an atmosphere of steam,
ing water, in a loosely closed vessel for 5 hours.

ats, to satisfactorily pass the requirements, shall remain firm and hard
no signs of distortion, checking, cracking, or disintegrating.

RIC ACID AND MAGNESIA. The cement shall not contain more than
anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

an or Slag Cements are not used extensively and never in important
heir manufacture and properties may be briefly described as follows:
ace basic slag is granulated by running it in a molten condition into
his accomplishes two objects. The slag is broken up into fine particles
udden chilling enhances its hydraulic properties. These particles
and ground with hydrated lime, in the proportion of from 15 to 25%
d lime and from 75 to 85% of granulated slag. Such cement, known
EMENT, is slow-setting and slow-hardening, and does not develop as

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the Standard Specifications for Cement adopted *August 16, 1909, by the Amer-*
ty for Testing Materials. *See, also, Chapter XXIV, page 912, where, in a*
eference is made to a Report of a Joint Conference, published April 28,
ending changes in the standard specifications for cement.

iron oxide. There are some Portland cements, however, which are
STAINING CEMENTS, and where care is used in their manufacture a
free or comparatively free from iron oxide, they cause no troub
the non-staining cements which have been extensively used for
which staining would be objectionable, is La Farge Cement, before
It is made at Teil, France, is light-colored and contains a small p
iron and soluble salts. There are other non-staining cements on
For setting stones, and in order to RETARD THE SETTING of the ceme
stones are well bedded, 1 part by volume of lime-paste is usually
4 parts of the cement.

Cost of Portland Cement. Portland cement can now (1915) b
in this country at prices ranging from 90 cents to $2.50 per barrel, fr
cars at the mills. The cost of the sacks and the freight are extra.
price for single barrels varies from about $2.00 to $2.50 per barrel.
the cost of cement in carload lots is about 85 cts per bbl at the mills
charge of 10 cts per bbl for bags is made when the cement is deliver
bags. The extra charge is 40 cts, if delivered in cloth, but the mills
40 cts when the bags are returned in good condition. There is a c
cts when the cement is furnished in wooden barrels and no allowa
for barrels returned. It is generally cheaper in the end to buy th
cloth bags and return the empty bags. For about 500 miles,
charges are about 40 cts per bbl of cement, making the total cost
this distance $1.25, when purchased in cloth bags and when the 40
are refunded. Testing costs from 3 to 5 cts per bbl, or from $5
carload. Unloading and storing near the station cost about 3 cts p
about 2 cts per bbl are usually added to the costs to allow for ha
returning empty sacks, and freight-charges for and damage to same.
costs about 5 cts per bbl per mile. The total cost, therefore, a
these average costs, is about $1.38 per bbl for the cement ready
mortar or concrete. (For Cost of Concrete, see page 249.)

Water Required in Mixing Cement Mortar. Good Portla
requires relatively little water to make a good mortar. Neat ceme
from 20 to 22% (by weight) of water to produce the normal co

ing of Portland cement mortars and concretes, it does not appear
hem. Thin coats of mortar, such as plaster, and troweled surfaces
a which free moisture is formed should not be applied in freezing
they are apt to scale. In general, it is undesirable to work with
concrete in freezing weather, as the difficulties of properly mixing
g the materials are then increased; it must be admitted, however,
sful work with Portland-cement mortar and concrete has been done
tures considerably below freezing.

ect of Salt in Mortar. When salt is added to the water of mix-
reezing-point is lowered, and, within certain limits, the freezing of
or concrete is prevented. The ultimate strength of mortar does not
be reduced when the amount of salt does not exceed 10%. Tetmajer
mount of salt required to lower the freezing-temperature as equal to
weight of the water per degree F. below 32°. The rule for the pro-
salt used in the works at Woolwich Arsenal, is said to have been as
Dissolve 1 lb of rock-salt in 18 gal of water when the temperature is
and add 3 oz of salt for every three degrees of lower temperature."

of Hot Water and of Soda. Hot water hastens the setting of
ement mortar, and 2 lb of carbonate of soda in 1 gal of water, boiled
in mortar, hastens the setting and lessens the danger of freezing.

y of Mortar required for Masonry and Plastering.* "One bbl
d cement and 3 bbl of sand, thoroughly and properly mixed, will
bbl, or 12 cu ft of good strong mortar. This will be sufficient to lay
yd of rough stone, or about 750 bricks, with from ¼ to ⅜-in joints,
15 sq ft of surface, 1 in thick, or 250 sq ft, ½ in thick."

l of natural cement and 2 bbl of lime, mixed with about ½ bbl of
make 8 cu ft of mortar, sufficient to lay 522 common bricks, with
⅜-in joints, or about 1 cu yd of rough rubble."

top coat of walks or floors, 1 bbl of Portland cement and 1 of sand
from 75 to 80 sq ft, ½ in thick, or from 50 to 56 sq ft, ¾ in thick.
of Portland cement and 1½ bbl of sand will cover from 110 to 120 sq
½ in thick, or from 75 to 80 sq ft, ¾ in thick.

Then a lever was rigged and the bolts pulled, with the following re
"Sulphur: Three bolts out of four developed their full strength
31 000 lb. One 1-in bolt failed by drawing out, under 12 000 lb. L
bolts out of four developed their full strength, as above. On
pulled out, under 13 000 lb. Cement: Five of the bolts out of six
out pulling out. One 1-in bolt began to yield in the cement at 26
sustained the load a few seconds before it broke.

"While this experiment demonstrated the superiority of cement,
strength and ease of application, it did not give the strength per squ
area. To determine this, four specimens of limestone were prep
10 in wide, 18 in long and 12 in thick, two of them having 1¾-in hol
of them 2¾-in holes drilled in them. Into the small holes 1-in
cemented, one of them being perfectly plain round iron, and the oth
thread cut on the portion which was embedded in the cement. Into
holes were cemented 2-in bolts similarly treated, and the four spec
allowed to stand 13 days before completing the experiment. At the
time they were put into a standard testing-machine and pulled. The
bolt began to yield at 20 000 lb and the threaded one at 21 000 lb.
plain bolt began to yield at 34 000 lb and the threaded one at 32
force in all cases being very slowly applied. The pump was then run
speed, and the stones holding the 2-in bolts split at 67 000 lb in the
smooth one and at 50 000 lb in the case of the threaded one.

"It is thus seen that for anchoring bolts in stone, cement is mo
stronger and easier of application than either lead or sulphur, and
sistance is from 400 to 500 lb per sq in of surface exposed. It is
ascertained fact that it preserves iron rather than corrodes it. T
used throughout the experiment was an English Portland cement."

6. Concrete †

Properties and Uses of Concrete.‡ There is probably no mat
so enduring or better adapted for foundations, walks and basement
than cement concrete, and for certain classes of buildings it is us
vantage for the walls, floors and interior supports. There are now th

of purposes is rapidly extending, due to the reduced price of
nt, and to a better appreciation and understanding of its proper-
ts. Concrete may be defined as an artificial stone, made by
t, water and what is called an aggregate, consisting of small and
of sand or screenings and gravel or broken stone; and when
od Portland cement, in proper proportions, it becomes so hard
at when pieces of it are broken, the line of fracture often passes
articles of stone, showing that the adhesion of the cement to the
r than the cohesive strength of the stone itself.

ates.* "Extreme care should be exercised in selecting the aggre-
tar and concrete, and careful tests made of the materials for the
etermining their qualities and the grading necessary to secure
sity or a minimum percentage of voids. A convenient coefficient
the ratio of the sum of the volumes of materials contained in a
o the total unit volume. (See, also, page 914.)

Aggregates should consist of sand, crushed stone, or gravel screen-
rom fine to coarse and passing when dry a screen having ¼-in diam
rably should be of siliceous material, and should be clean, coarse,
t, soft particles, vegetable loam or other deleterious matter, and
6% should pass a sieve having 100 meshes per lin in. Fine aggre-
always be tested. Fine aggregates should be of such quality
omposed of one part Portland cement and three parts fine aggre-
t when made into briquettes will show a tensile strength at least
strength of 1 : 3 mortar of the same consistency made with the
and standard Ottawa sand. This is a natural sand obtained at
assing a screen having 20 meshes and retained on a screen having
r lin in. It is prepared and furnished by the Ottawa Silica Com-
s per lb, free on board cars, at Ottawa, Ill., under the direction of
ommittee on Uniform Tests of Cement of the American Society
ineers. If the aggregate be of poorer quality the proportion of
d be increased in the mortar to secure the desired strength. If
developed by the aggregate in the 1 : 3 mortar is less than 70% of
of the Ottawa-sand mortar, the material should be rejected. To
oval of any coating on the grains, which may affect the strength,
ould not be dried before being made into mortar, but should con-
noisture. The percentage of moisture may be determined upon a
ple for correcting weight. From 10 to 40% more water may be
ixing bank or artificial sands than for standard Ottawa sand to
ame consistency.

e Aggregates should consist of crushed stone or gravel which is
screen having ¼-in diam holes and graded from the smallest to
articles; they should be clean, hard, durable and free from all
atter. Aggregates containing dust and soft, flat or elongated
ld be excluded from important structures."

f stone is suitable for the coarse aggregate which has such strength
gth of the concrete is not limited by the strength of the stone.
h is of little advantage beyond this minimum. The stones gener-
l are granites, traps and limestones. Shales and sandstones of

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e matter of this paragraph, and of following paragraphs relating to concrete,
and conclusions formulated by the joint committees *of the Am. Soc. C. E.,*
est. Mats., Am. Ry. Eng. *and Maint. of Way Asso., and Asso. of Am.*
nt Manfrs. *In regard to Aggregates, etc., see, also, the same subjects in*
pages 913 and 914, and foot-notes on page 913 in that chapter.

increased, as a large aggregate produces a stronger concrete tha
although it should be noted that the danger of separation from
becomes greater as the size of the coarse aggregate increases."

The use to be made of the concrete determines the maximum size o
aggregate. When used in mass-concrete construction, such as b
the maximum size may run up to 2½ and 3 in with good results. Fo
work and thin walls, however, it is necessary to reduce the maximu
in or less. It has been found that the following are the maximum
coarse aggregate of plain or mass-concrete in the best practice: for f
2½ in; for abutments, 2 in; for arch-rings, 1¼ in; and for copings,
etc., 1 in.

"Cinder concrete should not be used for reinforced-concrete str
may be allowable in mass for very light loads or for fire-protectio
The cinders used should be composed of hard, clean, vitreous clinke
sulphides, unburned coal, or ashes. (See, also, page 914.)

"**Water for Mixing Concrete.** The water used in mixing conc
be free from oil, acid, alkalies, or organic matter."

Preparing and Placing Mortar and Concrete. "(1) **Proport**
materials to be used in concrete should be carefully selected, of unifo
and proportioned with a view to securing as nearly as possible
density.

"(a) Unit of Measure. The unit of measure should be the cu
bag of cement, containing 94 lb, net, should be considered the equi
cu ft. The measurement of the fine and coarse aggregates should
volume.

"(b) Relation of Fine and Coarse Aggregates. The fine and c
gates should be used in such relative proportions as will insure ma
sity. In unimportant work it is sufficient to do this by individual
using correspondingly higher proportions of cement; for important
proportions should be carefully determined by density-experimen
sizing of the fine and coarse aggregates should be uniformly mainta
proportions changed to meet the varying sizes.

"(c) Relation of Cement and Aggregates. For reinforced-co
struction, one part of cement to a total of six parts of fine and c

ngs, thin walls, reinforced concrete, d impervious construction........	from 1 : 2 : 4 to 1 : 2½ : 4½
ures requiring great strength rather ss................................	from 1 : 2½ : 5 to 1 : 3 : 6
tures requiring mass rather than , foundations, etc..................	from 1 : 3 : 6 to 1 : 4 : 8

ng Concrete. The ingredients of concrete should be thoroughly e mixing should continue until the cement is uniformly distributed s is uniform in color and homogeneous. As the maximum density strength of a given mixture depend largely on thorough and com- it is essential that the work of mixing should receive special atten- e. Inasmuch as it is difficult to determine, by visual inspection, concrete is uniformly mixed, especially where limestone or aggre- the color of cement are used, it is essential that the mixing should inite period of time. The minimum time will depend on whether done by machine or hand.

suring Ingredients. Methods of measurement of the proportions is ingredients should be used which will secure separate and uni- ements of cement, fine aggregate, coarse aggregate and water at

hine-Mixing. When the conditions will permit, a machine-mixer ich insures the uniform proportioning of the materials throughout uld be used, as a more uniform consistency can be thus obtained. should continue for a minimum time of at least one minute after dients are assembled in the mixer.

l-Mixing. When it is necessary to mix by hand, the mixing should er-tight platform and especial precautions should be taken to turn dients together at least six times and until they are homogeneous in nd color."

atisfactory method * of mixing concrete by hand is to first prepare g of the materials, a tight floor of planks, or, better still, of sheet edges turned up about 2 in. Upon this platform should first be

stances should concrete be used that has partly set. A slow-se
should be used when a long time is likely to occur between mixing
Concrete should be deposited in such a manner as will permit the m
compacting, such as can be obtained by working with a straight sho
tool kept moving up and down until all the ingredients have set
proper places by gravity and the surplus water has been forced to
Special care should be exercised to prevent the formation of LAIT
hardens very slowly and forms a poor surface on which to deposit fr
All LAITANCE should be removed. When suspended work is re
crete previously placed should be roughened, thoroughly cleanse
material and laitance, thoroughly wetted and then slushed with a
sisting of one part Portland cement and not more than two parts fi
The faces of concrete exposed to premature drying should be ke
period of at least seven days."

"(b) Mixing and Depositing Concrete in Freezing Weathe
should not be mixed or deposited at a freezing temperature, unles
cautions are taken to avoid the use of materials covered with ic
containing frost, and to provide means to prevent the concrete f
after being placed in position and until it has thoroughly harde
coarse aggregate forms the greater portion of the concrete, it is
important that this material be heated to well above the freezing-p

"(c) Rubble Concrete. Where the concrete is to be deposite
work, its value may be improved and its cost materially reduced b
clean stones thoroughly embedded in the concrete and as near t
possible while still entirely surrounded by concrete.

"(d) Depositing Concrete Under Water. In placing concrete u
is essential to maintain still water at the place of deposit. The use
properly designed and operated, is a satisfactory method of plac
through water. The concrete should be mixed very wet (more s
dinarily permissible) so that it will flow readily through the trem
the places with practically a level surface. The coarse aggregat
smaller than ordinarily used, and never more than 1 in in diameter.
gravel facilitates mixing and assists the flow of concrete through
The mouth of the tremie should be buried in the concrete so tha

ost important considerations in the erection of a structure of con-
ced concrete. Care should be taken to inspect the concrete and
ardness before removing the forms. So many conditions affect
of concrete, that the proper time for the removal of the forms
led by some competent and responsible person, especially where
ic conditions are unfavorable. It may be stated, in a general
s should remain in place longer for reinforced concrete than for
e concrete, and that the forms for floors, beams and similar hori-
es should remain in place much longer than for vertical walls.
crete gives a distinctive ring under the blow of a hammer, it is
dication that it has hardened sufficiently to permit the removal
ith safety. If, however, the temperature is such that there is
that the concrete is frozen, this test is not a safe reliance, as
may appear to be very hard."

of Concrete and Temperature-Changes. "Shrinkage of con-
hardening and contraction from temperature-changes, causes
e of which depends on the extent of the mass. The resulting
portant in monolithic construction and should be considered care-
signer; they cannot be counteracted successfully, but the effects
ed. Large cracks produced by quick hardening or wide ranges
can be broken up to some extent into small cracks by placing
in the concrete; in long continuous lengths of concrete, it is
de shrinkage-joints at points in the structure where they will do
m. Reinforcement is of assistance and permits longer distances
age-joints than when no reinforcement is used. Small masses or
concrete should not be joined to larger or thicker masses without
hrinkage at such points. Fillets similar to those used in metal
f larger dimensions, for gradually reducing from the thicker to
dy, are of advantage. Shrinkage-cracks are likely to occur at
esh concrete is joined to that which is set, and hence in placing
construction-joints should be made on horizontal and vertical
ossible, at points where joints would naturally occur in dimen-
onry."

the metal in girders and columns be protected by a minimum of
crete; that the metal in beams be protected by a minimum of 1½ i
and the metal in floor-slabs be protected by a minimum of 1 in
It is recommended that in monolithic concrete columns, the concre
of 1½ in be considered as protective covering and not included in
section. It is recommended that the corners of columns, girders a
beveled or rounded, as a sharp corner is more seriously affected b
round one."

Waterproofing Concrete. "Many expedients have been us
concrete impervious to water under normal conditions, and also un
conditions that exist in reservoirs, dams and conduits of various k
rience shows, however, that where mortar or concrete is proportion
the greatest practicable density and is mixed to a rather wet con
resulting mortar or concrete is impervious under moderate press
crete of dry consistency is more or less pervious to water, and c
various kinds have been mixed with the concrete, or applied as a
surface for the purpose of making it water-tight. Many of these
are of but temporary value, and in time lose their power of imparti
ability to the concrete. In the case of subways, long retaining-wal
voirs, provided the concrete itself is impervious, cracks may be so
horizontal and vertical reinforcement properly proportioned and
they are too minute to permit leakage or are soon closed by infiltr
Coal-tar preparations applied either as a mastic or as a coating on
fabric are used for waterproofing, and should be proof against inju
or gases. For retaining-walls and similar walls in direct conta
earth, the application of one or two coatings of hot coal-tar pitch
oughly dried surface of concrete is an efficient method of preventi
tration of moisture from the earth." (See, also, Waterproofing
tions, Part III, pages 1629 to 1637.)

Surface-Finish of Concrete. "Concrete is a material of an ind
and should not be used in imitation of other structural materials.
important problems connected with its use is the character of t
exposed surfaces. The finish of the surface should be determined
concrete is placed, and the work conducted so as to make possib

r ordinary conditions this ratio should not exceed six. It rches of moderate span, where the conditions as to founda-

aterials Required per Cubic Yard of Concrete.* The the quantities of Portland cement required to make 1 cu yd uantities of cement, sand and stone required to make 1 cu ey are based upon formulas deduced by Halbert P. Gillette.

of Portland Cement per Cubic Yard of Mortar

, 35%, 1 bbl of cement yielding 3.65 cu ft of cement paste

ent to	1 to 1	1 to 1½	1 to 2	1 to 2½	1 to 3	1 to 4
	bbl	bbl	bbl	bbl	bbl	bbl
e 3.5 cu ft	4.22	3.49	2.97	2.57	2.28	1.76
e 3.8 cu ft	4.09	3.33	2.81	2.45	2.16	1.62
e 4.0 cu ft	4.00	3.24	2.73	2.36	2.08	1.54
e 4.4 cu ft	3.81	3.07	2.57	2.27	2.00	1.40
l per cu	0.6	0.7	0.8	0.9	1.0	1.0

of Portland Cement per Cubic Yard of Mortar

, 45%, 1 bbl of cement yielding 3.4 cu ft of cement paste

ent to	1 to 1	1 to 1½	1 to 2	1 to 2½	1 to 3	1 to 4
	bbl	bbl	bbl	bbl	bbl	bbl
e 3.5 cu ft	4.62	3.80	3.25	2.84	2.35	1.76
e 3.8 cu ft	4.32	3.61	3.10	2.72	2.16	1.62
e 4.0 cu ft	4.19	3.46	3.00	2.64	2.05	1.54
e 4.4 cu ft	3.94	3.34	2.90	2.57	1.86	1.40
d per cu	0.6	0.8	0.9	1.0	1.0	1.0

ables remember that the proportion of cement to sand is y weight. If the specifications state that a barrel of cement to hold 4 cu ft, for example, and that the mortar shall be

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ssion, from the Handbook of *Cost Data for Contractors and En-* Gillette, *published by The Myron C. Clark Publishing Company, revised edition, pages 538 to 540. This handbook* contains com- *a on quantities, costs, etc., of building*

of which the sand is composed. A quartz sand having 35% voids w
per cu ft; but a quartz sand having 45% voids weighs only 91 lb pe
the weight of the sand must be guessed at, assume 100 lb per cu
specifications require a mixture of 1 part of cement to 2 parts of sand,
we will have 380 lb (or 1 bbl) of cement mixed with 2 times 380, o
sand; and if the sand weighs 90 lb per cu ft, we shall have 760 divid
8.44 cu ft of sand to every barrel of cement. In order to use the t
given, we may specify our own size of barrel; let us say 4 cu ft; then, 8
by 4 gives 2.11 parts of sand by volume to 1 part of cement. Witho
error we may call this a 1 to 2 mortar, and use the tables, rememberi
barrel is now 'specified to be' 4 cu ft. If we have a brand of cement
3.4 cu ft of paste per bbl and sand having 45% voids, we find that app
3 bbl of cement per cu yd of mortar will be required.

"It should be evident from the foregoing discussions that no ta
made, and no rule can be formulated that will yield accurate results
brand of cement is tested and the percentage of voids in the sand d
This being so, the sensible plan is to use the tables merely as a r
and, where the quantity of cement to be used is very large, to make a
of mortar, using the available brands of cement and sand in the
specified. Ten dollars spent in this way may save a thousand, even
paratively small job, by showing what cement and sand to select."

Ingredients in One Cubic Yard of Concrete *

Sand-voids, 40%; stone-voids, 45%; Portland-cement barrel yielding 3.
paste. Barrel specified to be 3.8 cu ft

Proportions by volume	1 : 2 : 4	1 : 2 : 5	1 : 2 : 6	1 : 2½ : 5	1 : 2½
Barrels cement per cu yd concrete....................	1.46	1.30	1.18	1.13	1.0
Cubic yard sand per cu yd concrete....................	0.41	0.36	0.33	0.40	0.3
Cubic yard stone per cu yd					

l be, approximately, the number of barrels of cement per cubic

a 1 : 2 : 5 concrete, the sum of the parts is 1 plus 2 plus 5, which is
ivided by 8 is 1.25 bbl, which is approximately equal to the 1.30 bbl
table. Neither this rule nor this table is applicable if a different
it-barrel is specified, or if the voids in the sand or stone differ mate-
40% and 45% respectively. There are such innumerable com-
varying voids, and varying sizes of barrels, that the author does
worth while to give other tables."

Ingredients in One Cubic Yard of Concrete *

, 40%; stone-voids, 45%; Portland-cement barrel yielding 3.65 cu ft of paste. Barrel specified to be 4.4 cu ft

ons by volume	1 : 2 : 4	1 : 2 : 5	1 : 2 : 6	1 : 2½ : 5	1 : 2½ : 6	1 : 3 : 4
ment per cu yd	1.30	1.16	1.00	1.07	0.96	1.08
d sand per cu yd	0.42	0.38	0.33	0.44	0.40	0.53
l stone per cu yd	0.84	0.95	1.00	0.88	0.95	0.71
ions by volume	1 : 3 : 5	1 : 3 : 6	1 : 3 : 7	1 : 4 : 7	1 : 4 : 8	1 : 4 : 9
ment per cu yd	0.96	0.90	0.82	0.75	0.68	0.64
d sand per cu yd	0.47	0.44	0.40	0.49	0.44	0.42
l stone per cu yd	0.78	0.88	0.93	0.86	0.88	0.95

le is to be used when the cement is measured loose, after dumping it into a
r such conditions a barrel of cement yields 4.4 cu ft of loose cement.

Concrete. (For Cost of Cement, see page 238.) The average
may be taken at 30 cts per cu yd to cover digging and loading, but
d or screened the cost averages between 40 and 55 cts per cu yd.
l freight-charges generally raise the cost of sand, ready to unload at
from 90 cts to $1.10 per cu yd, and about 15 cts per yd additional
led, if unloaded from cars. Gravel costs from $1.20 to $1.40 per cu
d at the job, and crushed stone from $1.45 to $1.60. These prices
se, average prices only, and include moderate-haul teaming and un-
or hand-mixing and placing of soft concrete, and spreading without
ng, the labor-cost varies from 90 cts to $1.30 per cu yd. This is
g in barrows materials that are conveniently at hand. This cost
h higher for dry concrete, and hand-mixing costs may reach $2 or
d. For machine-mixing alone and with machines taking four bags
n, the cost of mixing may be even as low as 50 or 60 cts per cu yd.
alone, the cost is about 75 cts per cu yd; this *includes wheeling the*
mping it in place *and spreading and spading it into forms.* This
e almost doubled where unusual care had to be exercised to obtain
e and where there was an extra amount of spading. The costs

For floor-slabs not exceeding 4½ in in thickness. . . .	$1.00
For floor-slabs exceeding 5 in in thickness.	$1.00
For columns and thin walls.	$1.50
For walls exceeding 18 in in thickness.	$1.00
For dams and thick retaining-walls.	$0.70

For the unit cost due to the cost of the tools, plant and suppli taken as an average for jobs requiring from 4 000 to 10 000 cu y It varies, of course, with the character and magnitude of the wo for this item is reduced in larger jobs, falling to 80 or even 70 and it is increased in operations of less magnitude to from $1 to $1 for, say, 3 000 cu yd of concrete. When the amount of concrete small as 600 or 700 cu yd, hand-mixing is generally more ec machine-mixing. Mr. Allen summarizes * the cost of 1 cu yd of building requiring 5 000 cu yd of reinforced-concrete work in floor as follows, the cost of forms and steel and finishing of the surf included:

Cement, 1⅔ bbl, at $1.38 per bbl. . . . .
Sand, ½ cu yd, at $1 per cu yd. . . . .
Stone, 1.35 tons, at $1.40 per ton. . . . .
Labor, per cu yd. . . . .
Plant, per cu yd. . . . .

Total, per cu yd. . . . .

In this summary the exact theoretical proportions or quantit sand and stone required for 1 cu yd of concrete, and deduced f are not adhered to, the author stating that the exact theoretical p the net quantities of the materials determined by careful exp "conditions on actual construction work do not approach those work and that there is always a considerable waste of cement, san In view of these facts, he states that, "when estimating quantities to allow less than the following amounts of cement for differen of mix:

1 : 1½ : 3 mix . . . . 2.00 bbl pe

gth of Concrete. See Chapter V.

tamples of Portland-Cement Concrete. From the foregoing it
for foundation-work to-day, mass-concrete varies in proportions
: 6 to a 1 : 4 : 8 mix. Some of the earlier examples are added for

ns of the United States Naval Observatory, Georgetown, D. C.:
nt, 2½ sand, 3 gravel, 5 broken stone. (1 bbl of cement, 380 lb,
d of concrete.)

ns of the Cathedral of St. John the Divine, New York: 1 part
nent, 2 parts sand, 3 parts quartz gravel of pieces from 1½ to 2 in
(17 000 bbl of cement made 11 000 yd of concrete.)

n Life Insurance Building, New York, filling of caissons: 1 part
nd cement, 2 parts sand, 4 parts broken stone.

Building (15 stories), New York, filling of caissons: 1 part Portland
rts sand, 7 parts stone, finished on top for brickwork with 1 part
parts gravel.

Baker states that the concrete foundations under the Washington
vere made of 1 part Portland cement, 2 parts sand, 3 parts gravel
broken stone, and that this mixture stood, when six months old, a
lb per sq in, or 144 tons per sq ft.

BY
GRENVILLE TEMPLE SNELLING
MEMBER OF AMERICAN INSTITUTE OF ARCHITECTS

1. Mechanical Principles Involved

General Principles. Before discussing more in detail the prob
to masonry structures, in which, if improperly constructed, a tend
or overturn on their bases may be developed, a familiarity with wh
as the THEOREM OF FRICTION and the THEOREM OF THE MIDDLE T
of assistance in comprehending the methods indicated for renderin
tures stable.

Theorem of Friction. If a body rests on an inclined plane it
stationary until the angle ϕ, that the plane makes with the

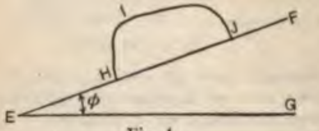


Fig. 1

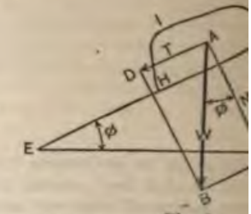
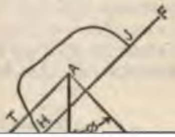
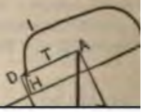


Fig. 2





can be resolved into two component forces, one, AC, normal to the ne and the other, AD, parallel to it. It is the parallel or tangential tends to pull the body down the plane and which is resisted by the reloped between the two surfaces. The friction developed between rfaces in contact depends upon the nature of the materials of which mposed and the intensity of the forces pressing them together; and he tendency to slide only up to a certain point. As the angle ϕ, inclined plane makes with the horizontal, increases, the tangential T, of the weight W, increases, until it becomes greater than the resistance, and the body moves down the plane (Fig. 3). From ry,

$$T = W \sin \phi$$

$$N = W \cos \phi, \quad \text{or,} \quad T = N \tan \phi$$

evidently a position of the plane, intermediate between the positions igs. 1 and 3, in which the component force T is just balanced by the d in which the body remains at rest although just on the point of ig. 4). If the angle which the inclined plane makes with the horithe moment when the body is just about to slide, be designated by ϕ, developed between the two surfaces will be equal to $N \tan \phi$, since, ngle of inclination of the plane to the horizontal is ϕ, the tangential of the weight just balances the friction. From the equation $T = N$ evident that the friction is directly proportional to N and to $\tan \phi$. hen known as the COEFFICIENT OF FRICTION and ϕ as the ANGLE OF , in the case of stone surfaces, it is often known as the ANGLE OF

owing Table I gives the average values of these constants as deterexperiment.

Table I. Coefficients and Angles of Friction

Kind of surface	Coefficient of friction, tan ϕ	Angle of friction, ϕ
limestone and marble:		
ssed upon soft dressed	0.70	35° 00′
ressed upon hard dressed	0.55	28 50
ressed upon soft dressed	0.65	33 00
ick or concrete:		
y upon masonry	0.65	33 00
y upon wood (with the grain)	0.60	31 00
y upon wood (across the grain)	0.50	26 40
y upon dry clay	0.50	26 40
y upon wet or moist clay	0.33	18 20
y upon sand	0.40	21 50
y upon gravel	0.60	31 00
e upon steel or iron	0.40	21 50
ne upon steel or iron	0.30	16 40

iscussion only the weight AB (Figs. 2, 3 and 4), of the body has been ; but the body might be subjected to the *action of other forces be*orce of gravity, in *which case these other forces would be combined eight in order to find the resultant, this resultant being again resolved ntial and a normal component. Since the angle* BAC is equal to the

angle *FEG* (Figs. 2, 3 and 4), given a certain normal pressure ex
body on the plane, the amount of the tangential pressure *T* depen
angle *FEG*. The problem in actual practice reduces itself to so a
conditions that no matter what the position of the plane may be,
which the resultant *W*, makes with the normal *N*, to the plane, will n
than the ANGLE OF FRICTION OF REPOSE.

Theorem of the Middle Third. When any surface is subje
sure from the action of any force or forces, this TOTAL PRESSURE
sidered as a SYSTEM OF AN INFINITE NUMBER OF PARALLEL FOR
unequal in intensity. These forces will have a RESULTANT, whose
DIRECTION and POINT OF APPLICATION can be determined, either gr
by moments, as explained in Chapter VI. The determination o
elements of this resultant force may at times become of the utmos
to the engineer.

Pressure of this nature is technically known as the STRESS to w
face in question is subjected. (See Chapter I.) When the INT
STRESS is not the same at different points of a surface, it is calle
STRESS, while if, on the contrary, its intensity remains the same at
of the surface, it is called a UNIFORM STRESS.

When a stress varies it may do so in one or two ways. It ma
FORMLY, that is to say, in a uniform manner, following some de
variation, so that, knowing this law, its intensity may be determi
given point of the surface; or NON-UNIFORMLY, following no la
stress varies in the former manner it is called a UNIFORMLY VAR
This is the case most frequently met with in engineering problems

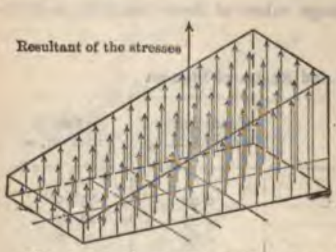


Fig. 5. Resultant within Middle Third

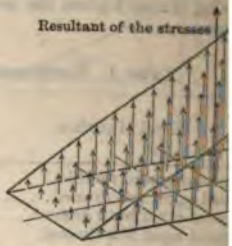


Fig. 6. Resultant at M

In dealing with ISOLATED FORCES, such as concentrated loads on
are usually interested in determining the MAGNITUDE and POINT OF
of the RESULTANT of these forces. When, however, the question is o
or of an unlimited number of forces, the problem that usually p
is one in which the resultant is known, in magnitude, direction
application, and in which it is required to determine the DISTRIBU
STRESS to which the surface is subjected. Or, in actual practice, i
to so arrange the parts of the structure that this resultant shall have
nitude, direction and point of application that the stress to which
under consideration is subjected shall not exceed certain LIMITS
determined beforehand by experience. For example, when the re
known amount of pressure or stress acts at the CENTER OF GRAVIT
face subjected to the stress, this stress is UNIFORMLY DISTRIBUT
surface.

the stress remaining the
point of application of
t is at a greater distance
dge than two-thirds the
of the surface, a certain
surface adjacent to the
st from the resultant is
a STRESS OF A CONTRARY
that distributed over

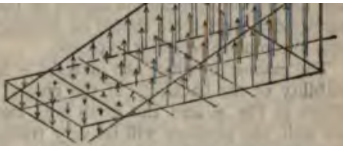


Fig. 7. Resultant beyond Middle Third

he area; that is to say, if the stress to which the major part of the
bject is a COMPRESSIVE stress, the stress acting on the remainder of
is a TENSILE stress. The stresses in a surface resulting from three
itions of the resultant force may be illustrated graphically, as shown
and 7.

2. Retaining-Walls

ns. A RETAINING-WALL is a wall built to resist the pressure of
or other filling or backing deposited behind it after it is built, as
l from a BREAST-WALL or FACE-WALL, which is a similar structure
vent the fall of earth which is in its undisturbed, natural position,
ich part has been excavated, leaving a vertical or inclined face. Fig
ration of the two kinds of wall.

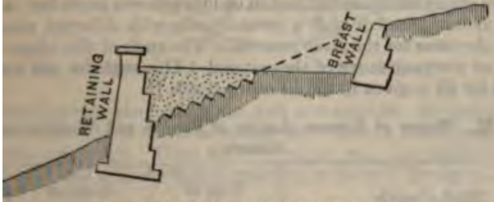


Fig. 8. Retaining-wall and Breast-wall

of Retaining-Walls. A great deal has been written on the THEORY
G-WALLS, and many theories, involving elaborate calculations for
the CONJUGATE PRESSURES in the earth-backing behind the wall,
leveloped for computing the THRUST which a bank of earth exerts
a wall, and for determining the FORM of wall which offers the great-
e with the least amount of material. There are so many condi-
er, upon which the thrust exerted by the backing depends, such as
of the earth, the dryness of the material, the mode of backing up
., that in practice it is impossible to *determine the exact thrust which*
ted against a wall of a given height. It is necessary, therefore, in
etaining-walls, to be guided by experience rather than by theory

include any in this work, but offer, rather, such suggestions, rules and
as have been established by practice and experience. A construct
gested from empirical data, which has been found to work well in pra
determining the THRUST OF THE EARTH-BACKING and the DIMENSIONS
WALL to properly resist this thrust, is given on page 257.

In designing a retaining-wall the backing as well as the wall itself
carefully considered. THE TENDENCY OF THE BACKING TO SLIP is ve
less when the material is in a dry state than when it is saturated wit
and hence every precaution should be taken to secure good drainage.
surface-drainage, there should be openings left in the wall for the wat
may accumulate behind it to escape.

The manner in which the material is filled against the wall, also, af
stability of the backing. If the ground is made irregular, with step
shown in Fig. 8, and the earth well rammed in layers inclined dow
the wall, the pressure will be very trifling, provided that attention i
drainage. If, on the other hand, the earth is tipped in the usual m
layers sloping DOWN TOWARDS the wall, almost the full pressure of t
will be exerted against it, and it must be made strong enough to w
such pressure.

Slopes of Repose and Angles of Repose. Cases may occur in p
which the conditions are not such as are shown in Fig. 8, which show
limited amount of fill or new material put in behind the wall on to
original slope of the grade; cases in which, on the contrary, the wall
built on the natural surface of the ground with a view to creating an
new terrace or embankment and where all the material back of the wa

All of this material does not beat upon the wall and tend to overtu
sand or loose earth taken from an excavation and deposited on the s
the ground does not spread itself out like a liquid but piles up in a moun
PILING UP is due to the FRICTION developed between the separate pa
they slide one over the other while being dumped. This phenomenon is
in the action of any solid material broken up into separate particles; and
the SLOPE OF THE SIDES of such a mound varies with different mater
in general, the same for the same material. The angle of this slope is
the ANGLE OF NATURAL SLOPE of the material. This angle for the mate
erally used for fill is given in the following Table II.

Table II. Slopes of Repose, Angles of Repose and Weights of
Materials

Kind of earth	Slope of repose*	Angle of repose	W lb
Sand, clean	1.5 to 1	33° 41′	
Sand and clay	1.33 to 1	36 53	
Clay, dry	1.33 to 1	36 53	
Clay, damp, plastic	2 to 1	26 34	
Gravel, clean	1.33 to 1	36 53	
Gravel and clay	1.33 to 1	36 53	
Gravel, sand and clay	1.33 to 1	36 53	
Soil	1.33 to 1	36 53	
Soft rotten rock	1.33 to 1	36 53	
Hard rotten rock	1 to 1	45 00	
Bituminous cinders	1 to 1	45 00	
Anthracite ashes	1 to 1	45 00	

* The slope is that of horizontal to vertical projection.

ERNAL ELASTICITY and tendencies toward a state of EQUILIBRIUM lay creating INTERNAL STRESSES which produce the CONJUGATE lready referred to. The exact determination of these INTERNAL nands relatively complicated calculations which would be out of ook of this character. The construction given in the following or determining the SLOPE OF THE CLEAVAGE-PLANE, between that acking which sustains itself and the triangular fill which actually wall, is sufficiently accurate, however, for all practical purposes.

of the Cleavage-Plane. The following construction (Figs. 9 d upon empirical data, for determining first, the PRISM OF EARTH

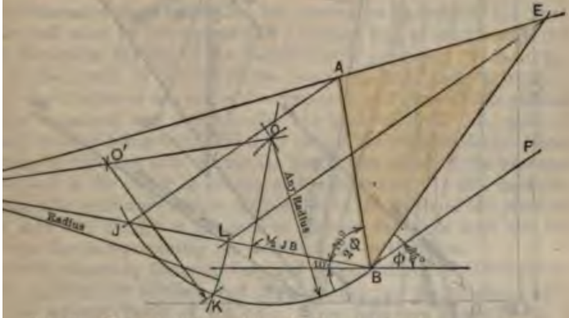


Fig. 9. Method of Determining the Prism of Earth

pressure on the back of the wall and secondly, the proper DIMEN- wall, has been found to work well in practice, when certain neces- ons are taken. These include proper DRAINAGE behind the wall, NG of the fill and efficient BRACING of the wall during its construc-

through *J* and *B*. Draw *HO* and bisect it, and with *O′* as a cente
a radius, describe the arc cutting the arc *JKB* at *K*. Again, with
and with *H* as center, describe the arc *KL*, and finally, from *L*, dra
to *JA*. The intersection of this line with the surface of the groun

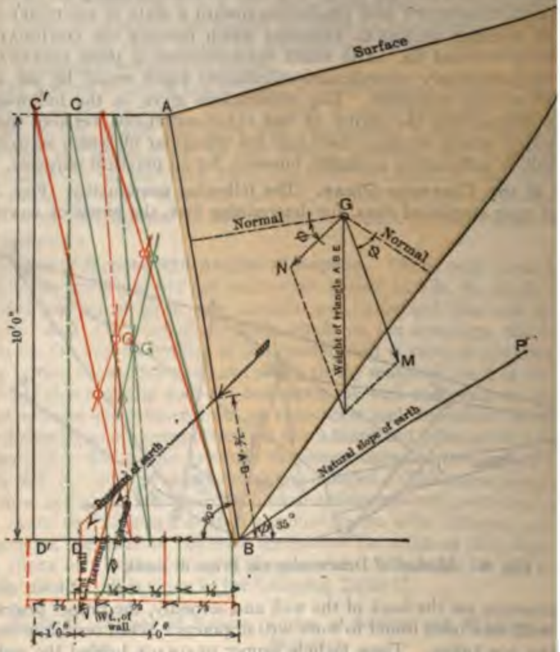


Fig. 10. Method of Determining Dimensions of Retaining-

point *E*. The line *EB* is the line of the CLEAVAGE-PLANE which
part of the backing which bears against the wall from the part wh
lateral pressure.

Having found the DIMENSIONS OF THE VOLUME OF EARTH, the th
must be resisted by the wall, the next step is to determine what the
OF THE WALL should be to properly resist this thrust. Usually one
are necessary before the proper solution of the problem is found.
ample given, a preliminary trial was made with a thickness at the
This construction is shown with the green lines (Fig. 10).

After drawing the triangle representing the base of the PRISM OF
its center of gravity, *G* (Chap. VI). From this point draw two nor
the back of the wall and the other to the line of the CLEAVAGE-PL
the two lines, *GM* and *GN*, making angles φ with these normals.
tically from the center of gravity, at any convenient scale of

he linear inch, the area of the triangle of the base of the prism, the
eady explained, being proportional to the volume of the prism and
Resolve this weight-line along the two lines *GM* and *GN* (Chap.
will give the MAGNITUDE and DIRECTION of the THRUST or pressure
against the wall. Apply this pressure at a point on the back of the
ird of the distance from the bottom, as shown by the arrow. This
which may tend to OVERTURN the wall and which tends to make it
the base.

these OVERTURNING and SLIDING-TENDENCIES, the weight of the wall
with the pressure of the earth behind it should produce a resultant
sfies the following conditions. First, its MAGNITUDE should not
nough to cause a unit pressure on the foundation-bed, greater than
y bear; secondly, it should pass within the MIDDLE THIRD of the base
stress over the entire area of the base will be a COMPRESSIVE stress;
, it should make an angle with a normal to the plane of the founda-
t greater than the ANGLE OF FRICTION between the stone, brickwork,
other masonry of the footings and the sand, clay, or rock of the foun-

to determine these conditions, the CENTER OF GRAVITY of the cross-
he wall must be determined and a vertical line drawn through this
it intersects the line of the EARTH-THRUST produced. It is at this
of the LINES OF ACTION of the two forces that their RESULTANT
nd the CENTER OF GRAVITY of the cross-section of the wall, the method
the trapezoid into two triangles has been followed, the center of
ach triangle being found and these two points being joined by a line.
ction of this line with the median line drawn between the base and
he wall is the center of gravity of the trapezoid. In this example,
ence, the scale used for the composition of the forces of the pressure
h and the weight of the wall is one-half the scale used for the
of the forces representing the weight of the earth-prism.

st trial, shown by the green lines, the first and third conditions neces-
ire stability are fulfilled; but the second is not, the resultant pass-
the MIDDLE THIRD of the base. This indicates, theoretically, a
LE stress or a tendency for the joints at the back of the wall to open.
al, therefore, is shown with the red lines, the thickness of the wall
ased as shown by the rectangle *CC'D'D*. In this second trial the
THE WALL is necessarily increased while the EARTH-THRUST remains
As in this case the resultant passes within the middle third, it is
hat a wall of these dimensions, 5 ft base by 10 ft height and with
ter, will be safe and will properly resist the thrust of the earth-

of Construction. Retaining-walls are generally built with a BATTER-
, a SLOPING face, as walls of this form are the strongest for a given
naterial; and if the courses are INCLINED DOWN TOWARDS THE BACK, the
slide on each other will be resisted, and it will not be necessary to
n the adhesion of the mortar. The importance of making the resist-
endent of the adhesion of the mortar is obviously very great, as it
wise be necessary to delay the backing up of the wall until the mortar
ghly set, which might require several months.

ork it is advisable to let every third *or fourth course below the frost-*
an inch or *two. This increases the friction of the earth against*
d causes the resultant of the forces acting behind the wall to become
ertical, and to fall farther within the base, increasing

It also conduces to strength to make the courses of varying heights thr the thickness of the wall, and to have some of the stones, especially th the back, sufficiently high to extend through two or three courses. means the whole masonry becomes more effectually interlocked or together as one mass and is less liable to bulge. The courses of mas often laid with their beds SLOPING IN, as in Fig. 15, to overcome the of the courses to slide on each other.

Where the ground freezes to a great depth, the back of the wall s SLOPED FORWARD for three or four feet below its top surface, as at *OC* (and this slope should be quite smooth, so as to lessen the hold of the prevent displacement.

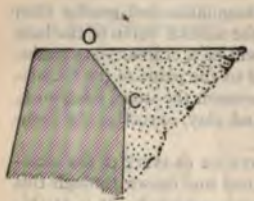


Fig. 11. Retaining-wall for Deep-freezing Earth

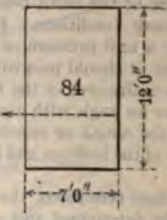


Fig. 12. Retaining-wall with Rectangular Cross-section

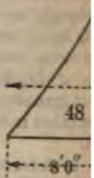


Fig. 13.
wall with
Cross-secti

Figs. 12, 13, 14 and 15 show the approximate RELATIVE VERTICAL SI AREAS of walls of different shapes that would be required to resist the of a bank of earth 12 ft high. The first three examples are calculated the maximum thrust of wet earth, while the last shows the modified form usually adopted in practice.

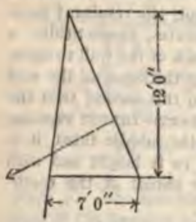


Fig. 14. Retaining-wall with Triangular Cross-section

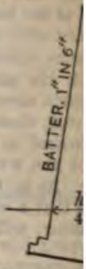


Fig. 15.
wall with
Back

Notes on the Thickness of Retaining-Walls. As has been stated, about the only practical rules for retaining-walls are the empirical rules based upon experience and tests. Trautwine* gives the following Table III for the thickness at the base of vertical retaining-walls with a sand backing deposited in the usual manner. The first column contains the vertical height *CD* (Fig. 16) of the earth as compared with the vertical height of the wall, *AB*. The latter is assumed to be 1, so that the table begin backing of the same height as the wall. These vertical walls may be to any extent not exceeding 1½ in to 1 ft, or 1 in 8, without affecti stability and without increasing the base.

If the wall is built as in Fig. 17, with the ground practically level top, the top of the wall should be not less than 18 in thick, and the thi at *a*, *a*, etc., just above each step, should be from one-third to two-fift

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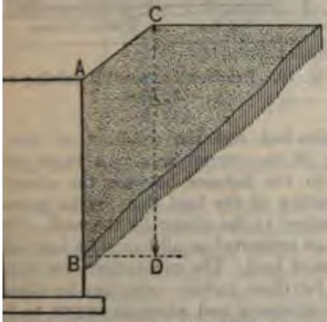
* The Civil Engineer's Pocket-Book, John C. Trautwine.

Table III. Proportions of Retaining-Walls

(Thickness of wall at the base in parts of the height, *AB*, Fig. 16)

height of the earth ared with the height wall above ground	Wall of cut stone in mortar	Wall of rubble or brick, good mortar	Wall of good, dry rubble
1	0.35	0.40	0.50
1.1	0.42	0.47	0.57
1.2	0.46	0.51	0.61
1.3	0.49	0.54	0.64
1.4	0.51	0.56	0.66
1.5	0.52	0.57	0.67
1.6	0.54	0.59	0.69
1.7	0.55	0.60	0.70
1.8	0.56	0.61	0.71
2	0.58	0.63	0.73
2.5	0.60	0.65	0.75
3	0.62	0.67	0.77
4	0.63	0.68	0.78
6	0.64	0.69	0.79
14	0.65	0.70	0.80
25	0.66	0.71	0.81
or more	0.68	0.73	0.83

from the top of the wall to each of these levels. If the earth is banked the top of the wall, the thicknesses should be increased as indicated by ble given above. If built upon ground that is affected by frost or surater, the footings should be carried sufficiently below the surface of the at the base to insure against heaving or settling.



16. Retaining-wall with Raised Sand Backing

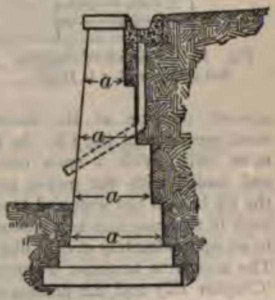


Fig. 17. Retaining-wall with Stepped Back

inforced-Concrete Retaining-Walls. With the constantly increasing REINFORCED CONCRETE for various purposes, there has come, *also*, the truction of retaining-walls in this material. *Figs.* 18,* 19* and 20* three designs by *A. L. Johnson for retaining-walls to satisfy* the requirements of banks 5, 10 and 20 ft high. The wall shown in F reinforced at intervals with COUNTERFORTS. The walls themselves in and 19 act as CANTILEVER BEAMS. The FOOTINGS, in all three c subjected to two principal external forces, the resultant of the

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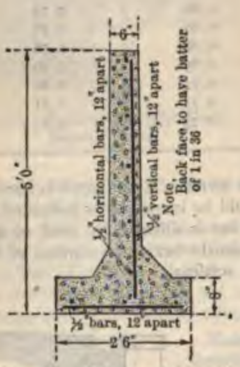
* *Plain and Reinforced Concrete, Taylor and Thompson.*



6"
5'0"
½" horizontal bars, 12" apart
½" vertical bars, 12" apart
Note,
Back face to have batter of 1 in 36
8"
½" bars, 12" apart
2'6"

Fig. 18. Reinforced-concrete Retaining-wall, 5 ft High

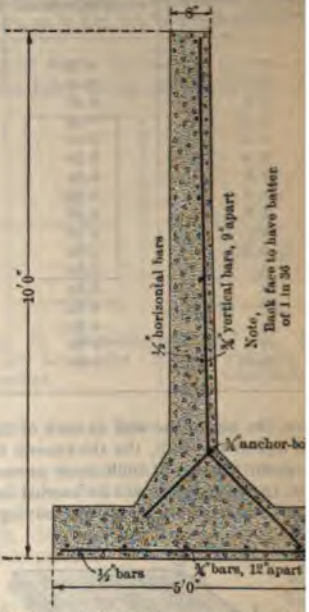


Fig. 19. Reinforced-concrete Retaining ft High

upward pressure of the foundation-bed and the resultant of t ward pressures of the fill. In Fig. 20 the COPING acts as a BEAM BOTH ENDS, with a span equal to the distance between the co and loaded with the proper proportion of the load due to the p the fill behind the wall and transmitted to the coping by the wall. itself in this case acts as a FLOOR-SLAB supported on all four sides and to an approximately evenly distributed load. The counterforts are i The MAXIMUM BENDING MOMENTS for these various cases can be d (Chapter IX) and the necessary DIMENSIONS and REINFORCEMENT vided decided by the rules given in Chapter XXIV.

3. Breast-Walls

Breast-Walls. Where the ground to be supported is firm, and the horizontal, the office of a BREAST-WALL (Fig. 8) is more to protect than the earth. It should be borne in mind that a trifling force skilfully app broken ground will keep in its place a mass of material, which, if once move, would crush a heavy wall. Great care, therefore, should be t e the newly opened ground to the influence of air and water longer than is ite for sound work, and to avoid leaving the smallest space for motion en the back of the wall and the ground. The strength of a breast-wall must oportionately increased when the strata to be supported incline down

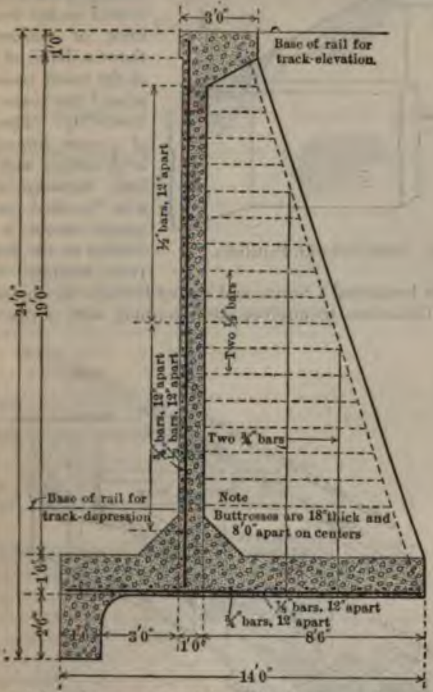


Fig. 20. Reinforced-concrete Retaining-wall with Counterforts and Apron

ds the wall; where they incline down from it, the wall need be little more a THIN FACING to protect the ground from disintegration. The preserva-f the NATURAL DRAINAGE is one of the most important points to be led to in the erection of breast-walls, as upon this their stability in a measure depends. No rule can be given for the best way to do this; it hatter for attentive consideration in each particular case.

4. Vault-Walls

ult-Walls. In large cities it is customary to utilize the space under the alk for storage or other purposes. This necessitates a wall at *the curb-* o hold back the earth and the street-*pressures and also the weight of the* alk. Where practicable *the space should be divided by partition-walls* t *every 10 ft. and when this is done the outer wall may be* advantageously f *hard bricks in the form of arches, as shown in Fig. 21.* The THICKNESS

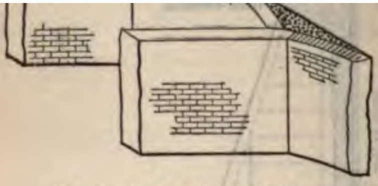
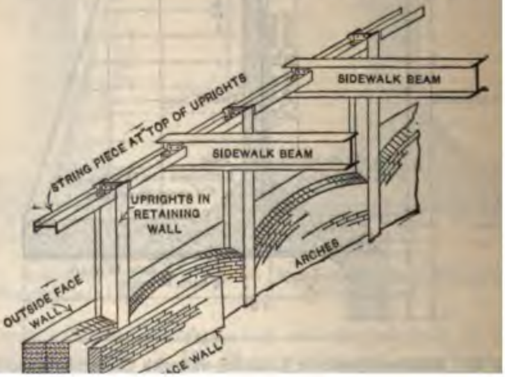


Fig. 21. Vault-wall with Partitions

shows a detail of the ou
of the vault under the s
around the Singer buildi
York City. These walls
of a core formed by t
brick arches with vertic
built between the fla
8-in vertical steel I
spaced about 5 ft ap
bedded at the bottom i
crete footing. Their t
joined by 6-in horizontal I beams and braced laterally by the sidewalk
5 ft apart. The arches themselves are segmental, with a rise of abo



CHAPTER V

TRENGTH OF BRICK, STONE, MASS-CONCRETE AND MASONRY

By

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1. Crushing Strength of Stonework, Brickwork, Bricks, etc.

Stresses in Masonry. By the term STRENGTH OF MASONRY is generally ant its resistance to a direct COMPRESSIVE force or load, and this is the only ect stress to which masonry should be subjected. Stone lintels and footings y be subjected to a TRANSVERSE or BENDING STRESS, but they can hardly be luded in the term masonry, as they consist of single pieces. There are also dencies to bend and to split apart in brick walls and piers, as they are usually h in proportion to their lateral dimensions, but the stresses thus developed not be accurately determined and should be avoided as much as possible. impossible to fix values for the strength of brickwork or stonework with hing like the exactness possible for wooden or steel members, for the reason there is not only a great variation in the strength of different kinds of and stone, even when taken from the same kiln or quarry, but the strength ls and piers is also greatly affected by the kind and quality of the mortar he way in which the work is built and bonded, and the amount of moisture materials when they are laid. All that can be done, therefore, is to give which will be safe for the different kinds of masonry built in the usual .

ing Compressive Strength of Masonry. The building laws of most rger cities of this country specify the maximum loads per square foot o be placed upon different kinds of masonry, and these laws must govern ects in such cities. When there is no restriction of this kind, Table I etty good idea of the maximum loads which it is safe to put upon the inds of work mentioned. Table II gives the maximum safe loads the building laws of several cities, and the remaining tables of the e records of numerous tests made to determine the ultimate com- engths of various kinds of bricks, building stones, mortars and con- are of value in determining the safe loads for special cases. In the safe compressive resistance of masonry from tests on the ulti- ssive strength of work of the same kind, a factor of safety of at d be allowed for piers and 20 for arches.

Table I. Safe Working Loads for Masonry

BRICKWORK IN WALLS OR PIERS

	Tons per square foot Eastern	 Western
e mortar	7	5
draulic-lime mortar	...	6
ural-cement mortar, 1 : 3	*10*	8
ricks in lime mortar	*8*	
cks in natural-cement mortar		
ts in Portland-cement m		

Piers exceeding in height six times their least lateral dimensions shoul increased 4 in in lateral dimensions for each additional 6 ft.

STONEWORK

	Tons squar
Rubble walls, irregular stones	
Rubble walls, coursed, soft stone	
Rubble walls, coursed, hard stone	5 to
Dimension-stone, squared, in cement mortar:	
Sandstone and limestone	10 to
Granite	20 to
Dressed stone, with ⅜-in dressed joints, in Portland-cement mortar:	
Granite	
Marble or limestone, best	
Sandstone	

The height of columns should not exceed eight times the least diameter the least diameter is sufficiently greater than necessary for the strength material used.

CONCRETE *

Portland-cement mortar, 1 : 8, 6 months, 10 tons; 1 year, 15 to 20

Natural-cement mortar, 1 : 6, 6 months, 3 tons; 1 year, 5 to 8 tons

HOLLOW TILE

Safe loads per square inch of effective bearing parts

Hard fire-clay tiles	80 l
Hard ordinary clay tiles	60 l
Porous terra-cotta tiles	40 l

MORTAR

In ½-in joints, 3 months old

	Ton squar
Portland-cement mortar, 1 : 4	
Natural-cement mortar, 1 : 3	
Lime mortar, best	8 to
Best Portland-cement mortar, 1 : 2, in ¼-in joints for bedding iron plates	

The values given above are generally very conservative. The leadin tects and engineers of Chicago recommended for that city in 1908 the ing SAFE WORKING PRESSURES for brick and stone masonry and concrete

Common brick of crushing strength equal to 1 800 lb per sq in:	Lb per sq in	To
In lime mortar	100	
In lime-and-cement mortar	125	
In natural-cement mortar	150	
In Portland-cement mortar	175	

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* See pages 283 to 287.

† These loads are allowed by the Chicago Building Ordinance.

	Lb per sq in	Tons per sq ft
d, common brick, of crushing strength equal oo lb per sq in:		
t Portland cement, ı lime-paste and 3 sand.	175	12⅗
Portland-cement mortar..................	200	14⅖
d sewer-brick, of crushing strength equal to lb per sq in:		
Portland-cement mortar..................	250	18
ck, in 1 : 3 Portland-cement mortar.......	350	25⅕
natural cement, 1 : 2 : 5.................	150	10⅘
Portland cement, 1 : 3 : 6, machine-mixed..	300	21⅗
Portland cement, 1 : 3 : 6, hand-mixed.....	250	18
Portland cement, 1 : 2 : 4, machine-mixed..	400	28⅘
Portland cement, 1 : 2 : 4, hand-mixed.....	350	25⅕
coursed, in lime mortar..................	60	4⅓
coursed, in Portland-cement mortar.......	100	7⅕
ursed, in lime mortar....................	120	8½
ursed, in Portland-cement mortar.........	200	14⅖
estone, in Portland-cement mortar........	400	28⅘
nite, in Portland-cement mortar..........	600	43⅕

Table II. Comparison of Building Laws

Materials	Boston, 1909	Buffalo, 1909	New York, 1906	Chicago, 1914	St. Louis, 1907	Philadelphia, 1914	Denver, 1898
	Allowable pressures in tons per sq ft						
....	60	....	72-172	43	....	....	40
limestone, cut..........	40	....	50-165	29	....	....	....
hard, cut...............	30	....	28-115	29	....	....	12
d brick in Portland- ortar................	20	12	15	12½	21½	....	....
d brick in natural- ortar................	18	9	....	10	....	15	9
d brick in cement-and- ar...................	12	....	11½	....	11½	12	....
d brick in lime mortar.	8	6	8	6½	11	8	8
k in Portland-cement	....	12	....	....	....	....	....
ck in natural-cement	....	9	....	....	....	....	12
ne in natural-cement	....	5*	8†	....	....	10	12
nent concrete in foun- : 2 : 4...............	30	4	16.5	18-28‡	18	15	10
ent concrete in foun- : 2 : 4...............	....	....	9	10½§	....	....	4

d-cement mortar.
nd-cement mortar, 10; in lime-cement mortar, 7; in lime mortar, 5.
to mixture.
iixture.

s. *As a rule brickwork is subject to its full safe resistance only iers, and in small sections of walls, under bearing-plates.* In the

Formulas for the Safe Strength of Brick Piers exceeding six in height. From the records of numerous tests on the strength of b from some formulas published* by Ira O. Baker, and also from perso vation, Mr. Kidder deduced the following formulas for the maximu loads for first-class brickwork in piers whose height exceeds six time lateral dimension.

For piers laid with rich lime mortar:

$$\text{Safe load per square inch} = 110 - 5\,H/D$$

For piers laid with 1 : 2 natural-cement mortar:

$$\text{Safe load per square inch} = 140 - 5\tfrac{1}{2}\,H/D$$

For piers laid with 1 : 3 Portland-cement mortar:

$$\text{Safe load per square inch} = 200 - 6\,H/D$$

H representing the height in feet, and D the least lateral dimension in

For a pier 20 ft high and 2 ft square these formulas will reduce th to 4.3 tons per sq ft for lime mortar, 6.1 tons for natural-cement 10 tons for Portland-cement mortar. No pier over 8 ft high should b 12 by 12 in in cross-section and when from 6 to 8 ft high piers should 8 by 12 in in cross-section.

The following is the Chicago law (1914): "Isolated piers of conc or masonry shall not be higher than six times their smallest dimensi the above unit stresses ‡ are reduced according to the following form

$$P = C\,(1.25 - H/20\,D)$$

in which P is the reduced allowed unit load, C the unit stress abov to, H the height of the pier in feet and D the least dimension of the p No pier shall exceed in height twelve times the least dimension. T of the pier shall be added to other loads in computing the load on th

ielves, and not by direct crushing of the bricks or mortar, showing
ıre weakest in their bond and in the tensile or transverse strengths of
It is very important, therefore, to have the brickwork well bonded,
ıts filled with mortar or grouted. The strength of a brick pier in-
:arry an extreme load would probably be increased by bonding fre-
:h hoop-iron in addition to the regular brick-bond.*

:ones in Brick Piers. Many competent architects and builders
at the strength of a brick pier is increased by inserting bond-stones,
in in thickness and the full size of the pier in cross-section, every 3 or
ht.

nple, the Building Laws for the City of New York (1906) require
s every 30 in in height, and at least 4 in in thickness, to be built into
which contain "less than 9 superficial feet at the base, and which sup-
eam, girder, arch, or column on which a wall rests, or lintel spanning
over 10 ft and supporting a wall." The New York laws allow cast-
of sufficient strength and of the full cross-section of the pier to be
d of the bond-stones. On the other hand, there are many first-class
ıo consider that bond-stones in a brick pier do more harm than good,
thor is of the opinion that this is generally the case. The Boston
aws do not require intermediate bond-stones. If bond-stones are
should be bedded so as to bear rather more heavily on the inner
the pier than on the outer 4 in, for unless this is done the outer shell
ıost of the load, and will be likely to bulge away from the core. A
supports a girder or column should have a cap-stone or iron plate of
:rength to distribute the pressure over the entire cross-section of the

ıced with Stone, Terra-Cotta, or Cement Blocks. Brick walls
blocks or ashlar of any material should always have the backing laid
mortar or in cement-and-lime mortar, unless the backing is very
is, 30 in or more. The aggregate thickness of the mortar joints in
; is so much greater than in the facing, that any shrinkage or com-
the mortar tends to throw undue weight on the facing and to sepa-
ı the backing. Veneering of any kind should be tied to the backing
ery 18 in in height. The New York Building Code (1906, Sections 28
juires that all bearing walls faced with bricks laid in running bond,
ls faced with stone ashlar less than 8 in thick, shall be of such thick-
make the wall independent of the facing conform to that required
l walls. Ashlar 8 in thick and bonded into the backing may be
part of the thickness of the wall.

g.† It is contended by persons having large experience in building
ıry carefully grouted, when the temperature is not lower than 40° F.,
ıe most efficient result. Many of the largest buildings in New York
grouted walls. The Mersey docks and warehouses at Liverpool,
ne of the greatest pieces of masonry in the world, were grouted
. It should be stated, however, that there are many engineers and
do not believe in grouting, claiming that the materials tend to
ıd form layers.

g Height of Brick and Stone. If we assume that the weight of
is 120 lb per cu ft, and that it would commence *to crush under 700 lb*

ınner in *which brick piers fail is excellently shown by* illustrations on page 79 per sq in, then a wall of uniform thickness would have to be 840 ft hi the bottom courses would commence to crush from the weight of the b above. Average sandstones, at 145 lb per cu ft, would require a column high to crush the bottom stones, and an average granite, at 165 lb p would require a column 10 470 ft high. The Merchants' shot-tower more is 246 ft high, and its base sustains a pressure of 6½ tons per sq ft being long tons of 2 240 lb. The base of the granite pier of Saltash B Brunel), of solid masonry to the height of 96 ft, and supporting the en iron spans of 455 ft each, sustains 9½ tons per sq ft.

Stone Piers. Piers of good strong building stone laid in course cross-sections of the piers, with the top and bottom courses bedded even, may be built to support very heavy loads. The height of s however, should not exceed ten times the least lateral dimension, an exceeds eight times the thickness, the load should be reduced. T should not exceed ⅜ in in thickness and should be spread with 1 : 2 cement mortar, kept back 1 in from the face of the pier to prevent s the edges. A test of the strength of a limestone pier 12 in square is under Marbles and Limestones, in this chapter. Rubble-work shou used for piers whose height exceeds five times the least dimension, or the latter is less than 20 in.

Records of Tests on the Crushing Resistance of Bricks. gives the results of some tests on bricks, made under the direction of M in behalf of the Massachusetts Charitable Mechanics' Association.

Table III. Ultimate and Cracking Strengths of Bricks

Kind of brick	Size of test-specimen	Area of face, sq in	Commenced to crack under lb per sq in
Philadelphia face-brick............	Whole brick	33.7	4 303
Philadelphia face-brick............	Whole brick	32.2	3 400
Philadelphia face-brick............	Whole brick	34.03	2 879
Average........................	....	....	3 527
Cambridge brick (Eastern)........	Half brick	10.89	3 670
Cambridge brick (Eastern)........	Whole brick	25.77	7 760
Cambridge brick (Eastern)........	Half brick	12.67	3 393
Cambridge brick (Eastern)........	Half brick	13.43	3 797
Average........................	....	....	4 655
Boston Terra-Cotta Co.'s brick....	Half brick	11.46	11 518
Boston Terra-Cotta Co.'s brick....	Whole brick	25.60	8 593
Boston Terra-Cotta Co.'s brick....	Whole brick	28.88	3 530
Average........................	....	....	7 880
New England pressed brick........	Half brick	12.95	3 862
New England pressed brick........	Half brick	13.2	8 180
New England pressed brick........	Half brick	13.30	2 480
New England pressed brick........	Half brick	13.45	4 535
Average........................	....	....	4 764

ens were tested in the government testing-machine at Watertown,
eat care was exercised to make the tests as perfect as possible.
l plates between which the bricks are crushed are fixed in one posi-
essary that each specimen tested should have perfectly parallel
ricks which were tested were rubbed on a revolving bed until the
m faces were perfectly true and parallel. The preparation of the
vay required a great deal of time and expense; and it was so difficult
ne of the harder bricks that they had to be broken and only one-half
repared at a time.

elphia bricks used in these tests were obtained from a Boston
ere fair samples of what is known in Boston as Philadelphia Face-
were very soft bricks.

idge bricks were the common bricks, such as are made around
y are about the same as the Eastern bricks.

Terra-Cotta Company's bricks were manufactured of a rather
were such as are often used for face-bricks.

England pressed bricks were hydraulic-pressed bricks, and were
d as iron.

made on the same machine by the United States Government in
erage strength of three (M. W. Sands) Cambridge, Mass., face-
925 lb, and of his common bricks, 18 337 lb per sq in, one brick
e enormous strength of 22 351 lb per sq in. This was a very hard-
Three bricks of the Bay State (Mass.) manufacture showed an
gth of 11 400 lb per sq in. The New England bricks are among
nd strongest in the country, those in many parts of the West not
urth the strength given above; so that in heavy buildings, where
f the bricks to be used is not known by actual tests, it is advisable
ricks tested. Ira O. Baker reported some tests on Illinois bricks,
100 000-pound testing-machine at the University of Illinois in
, which give for the crushing strength of soft bricks, 674 lb per sq in,
ge of three face-bricks, 3 070 lb per sq in, and for four paving-
b per sq in. In nearly all makes of bricks it will be found that the
e not as strong as the common bricks.

the Strength of Brick Piers Laid with Various Mortars.*
ere made for the purpose of testing the strength of brick piers
different cement mortars, as compared with those laid up with
tar. The bricks used in the piers were procured at M. W. Sands's
mbridge, Mass., and were good ordinary bricks. They were from
as the samples of common bricks described above. The piers
n in cross-section, and nine courses, or about 22½ in high, except-
which was but eight courses high. They were built Nov. 29, 1881,
storehouses at the United States Arsenal in Watertown, Mass. In
the two ends of the piers perfectly parallel surfaces, a coat of pure
ent, about ½ in thick, was put on the top of each pier and the foot
in the same cement. On March 3, 1882, three months and five
e tops of the piers were dressed to plane surfaces at right-angles
f the piers. On attempting to dress the lower ends of the piers,
out peeled off, and it was necessary to remove it entirely and put
cement similar to that on the tops of the piers. This was allowed
one month and sixteen days, when *the piers were tested. At that*
were four months and twenty-six days old. As the piers were
eather, the bricks were not wet. They were built by a skilled

usually, although not always, containing a low percentage of t
sets rapidly, has about one-third the strength of true Portland c
much weakened by the addition of sand.

Table IV. Tests of Piers of Common Bricks Laid in Differe

Piers 8 by 12 in in section, built of common bricks in common mortar	Ultimate strength of pier, lb	Pressure per sq in under which pier commence to crack, lb
Lime mortar	150 000	833
Lime mortar, 3 parts; Portland cement, 1 part	290 000	1 875
Lime mortar, 3 parts; Newark and Rosendale cements, 1 part	245 000	1 354
Lime mortar, 3 parts; Roman cement, 1 part	195 000	1 041
Portland cement, 1 part; sand, 2 parts	240 000	1 302
Newark and Rosendale cements, 1 part; sand, 2 parts	205 000	708
Roman cement, 1 part; sand, 2 parts	185 000	1 770

As the actual strength of brick piers is a very important con
building-construction, some tests, made by the United States Go
Watertown, Mass., and contained in the report of the tests made or
ment testing-machine for the year 1884, are given as being of
Three kinds of bricks were represented in the construction of t
mortars of different composition, ranging in strength from lime m
Portland-cement mortar. The piers ranged in cross-section dim

Number of test	Height		Cross-section	Composition of mortar	cubic foot	area	crack	Total	Lb per sq in	Tons per sq ft	Per cent of single brick
	ft	in			lb	sq in	lb	lb			
				Built of face-bricks (M. W. Sands, Cambridge, Mass.)							
11	1	4	8 by 8	1 lime mortar, 3 sand	137.4	57.00	85 000	143 600	2 520	181.4	18.1
320	6	8	8 by 8	1 lime mortar, 3 sand	133.5	57.76	50 000	108 400	1 877	135.1	13.5
12	1	4	8 by 8	1 Portland-cement mortar, 2 sand	136.3	57.76	200 000	218 100	3 776	271.8	27.1
321	6	8	8 by 8	1 Portland-cement mortar, 2 sand	133.5	57.76	85 000	129 900	2 249	161.9	16.2
283	2	0	12 by 12	1 lime mortar, 3 sand	....	132.25	140 000	257 100	1 940	139.7	13.9
284*	2	0	12 by 12	1 lime mortar, 3 sand	....	113.76	90 000	226 100	1 990	143.3	14.3
332	10	0	12 by 12	1 lime mortar, 3 sand	131.7	132.25	70 000	199 800	1 511	108.8	10.9
334†	10	0	12 by 12	1 lime mortar, 3 sand	125.0	115.44	100 000	208 600	1 807	130.1	13.0
286	2	0	12 by 12	1 Portland-cement mortar, 2 sand	....	132.25	200 000	486 000	3 670	264.2	26.4
326	10	0	12 by 12	1 Portland-cement mortar, 2 sand	132.2	132.25	200 000	298 000	2 253	162.2	16.2
				Built of common bricks (M. W. Sands)							
10	1	4	8 by 8	1 lime mortar, 3 sand	135.6	60.80	66 000	148 800	2 440	175.6	13.3
12½	6	8	8 by 8	1 lime mortar, 3 sand	133.6	62.40	....	96 100	1 540	110.8	8.4
281	2	0	12 by 12	1 lime mortar, 3 sand	....	138.06	75 000	296 400	2 150	154.8	11.7
282‡	2	0	12 by 12	1 lime mortar, 3 sand	....	119.58	120 000	244 600	2 050	147.6	11.2
331	9	9	12 by 12	1 lime mortar, 3 sand	131.5	138.06	70 000	154 300	1 118	80.5	6.1
330§	10	0	12 by 12	1 lime mortar, 3 sand	136.0	115.50	70 000	183 300	1 587	114.3	8.6
329	10	0	12 by 12	1 Portland-cement mortar, 2 sand	131.0	138.06	....	276 600	2 003	144.2	10.9
387	2	8	16 by 16	1 Portland-cement mortar, 2 sand	....	256.00	460 000	696 000	2 720	195.8	14.8
328	10	0	16 by 16	1 Portland-cement mortar, 2 sand	....	256.00	340 000	483 100	1 887	135.8	10.3

* Has a hollow core, 4.25 by 4.25 in. † Has a hollow core, 4.1 by 4.1 in. ‡ Has a hollow core, 4.5 by 4.5 in. § Has a hollow core, 4.75 by 4.75 in.

Table V (Continued). Tabulated Results of the Actual Crushing Strength of Brick Piers

Number of test	Nominal dimensions			Composition of mortar	Weight per cubic foot	Sectional area	First crack	Ultimate strength			
	Height		Cross-section					Total	Lb per sq in	Tons per sq ft	Per cent of single brick
	ft	in			lb	sq in	lb	lb			
				Built of common bricks (Bay State)							
285	2	0	12 by 12	1 lime mortar, 3 sand	....	146.41	95 000	201 000	1 370	98.6	12.0
288	6	0	12 by 12	1 lime mortar, 3 sand	....	144.00	70 000	163 200	1 133	81.6	9.9
289	6	0	12 by 12	1 lime mortar, 3 sand	119.7	144.00	100 000	174 300	1 210	87.1	10.6
291*	6	0	12 by 12	1 lime mortar, 3 sand	118.2	144.00	80 000	191 600	1 331	95.8	11.7
292†	6	0	12 by 12	1 lime mortar, 3 sand	118.1	156.25	110 000	189 200	1 211	87.2	10.6
325	7	10	12 by 12	1 lime mortar, 3 sand	120.0	144.00	100 000	169 100	1 174	84.6	10.3
327	10	0	12 by 12	1 lime mortar, 3 sand	118.0	144.00	90 000	133 100	924	66.6	8.1
335	10	0	8 by 12	1 lime mortar, 3 sand	107.0	96.00	35 000	90 200	940	67.7	8.2
333	10	0	12 by 16	1 lime mortar, 3 sand	118.7	192.00	80 000	148 500	773	55.7	6.8
301	6	0	12 by 12	1 Rosendale-cement mortar, 2 lime mortar........................	120.6	144.00	160 000	237 000	1 646	118.5	14.4
293	6	0	12 by 12	1 Rosendale-cement mortar, 2 sand	123.0	144.00	260 000	284 000	1 972	142.0	17.3
300	6	0	12 by 12	1 Portland-cement mortar, 2 lime mortar........................	120.3	144.00	150 000	203 200	1 411	101.6	12.4
294	6	0	12 by 12	1 Portland-cement mortar, 2 sand	119.7	144.00	220 000	258 000	1 792	129.0	15.7
290	6	0	12 by 12	Neat Portland-cement mortar	126.6	144.00	280 000	342 000	2 375	171.0	20.8

* Joints broken every six courses.

† Bricks laid on edge.

Tests of Brick Piers, McGill University Laboratories, March, 1897

sions of ers	Composition of mortar	Kind of bricks	Crushing strength, lb per sq in		Age
			At first crack	Maximum load	
1 in, 11.6 n; joints hick	1 Canadian Portland-cement mortar, 3 sand	Ordinary well-burned flat bricks	822	1 234	3 weeks
1 in, 11.6 n; joints hick	1 German Portland-cement mortar, 3 sand	Ordinary well-burned flat bricks	990	1 230	3 weeks
3 in, 10.5 n; joints hick	1 English Portland-cement mortar, 3 sand	La Prairie pressed bricks, keyed on one side	1 130	1 524	3 weeks
in, 10.75 ; joints hick	1 Belgian Portland-cement mortar, 3 sand	La Prairie pressed bricks, keyed on one side	1 204	1 985	3 weeks

VII. Tests of Brick Piers, Made at the University of Illinois

The amounts given are average values

teristics of piers	Average unit load lb per sq in	Ratio of strength of pier to strength of brick	Ratio of strength of pier to strength of first of series	Crushing strength of 6-in mortar-cubes lb per sq in	Ratio of strength of pier to strength of cubes
	Shale building bricks				
, 1 : 3 Portland- mortar, 67 days	3 363	0.31	Standard 1.00	2 870*	1.17
l, 1 : 3 Portland- mortar, 6 months	3 950	0.37	1.18	....	....
l, 1 : 3 Portland- mortar, eccentri- aded, 68 days	2 800	0.26	0.83	....	....
id, 1 : 3 Portland- mortar, 67 days	2 920	0.27	0.87	2 870*	1.05
l, 1 : 5 Portland- mortar, 65 days	2 225	0.21	0.66	1 710	1.30
d, 1 : 3 natural- mortar, 67 days	1 750	0.16	0.52	305	5.75
, 1 : 2 lime mortar, s	1 450	0.14	0.43	....	....
	Underburned clay bricks				
d, 1 : 3 Portland- t mortar, 63 days	*1 060*	*0.27*	*0.31*	*2 870**	*0.37*

value based on thirteen tests of 1 : 3 Portland-cement mortar-cubes, 60 days

[illegible]
clay bricks, crushing strength, flatwise, 3 900 lb per sq in; modulus of 480 lb per sq in.

Tensional Strength of Brickwork. For the safe tensional str brickwork and other materials in footings, see Chapter II, page 179.

2. Strength of Terra-Cotta and Terra-Cotta Piers

General Properties of Terra-Cotta. The lightness of terra-cotta, c with its great compressive strength, together with its durability and inc bility by fire, water, frost, etc., renders it an especially valuable building Terra-cotta for building purposes, whether plain or ornamental, is made of hollow blocks formed with webs to give extra strength and work true while drying. This is necessary because good, well-burn cotta cannot safely be made more than about 1½ in thick, whereas, quired to bond with brickwork, it must be at least 4 in thick. Wh strength is needed, these hollow spaces are filled with concrete or br which greatly increases the crushing strength of the terra-cotta, althou it is able to bear a very heavy weight. "A solid cubical block of te has borne a crushing-stress of more than 500 tons."

Crushing Strength of Terra-Cotta Blocks. Some exhaustive exp made by the Royal Institute of British Architects give the following r the crushing strengths of terra-cotta blocks:

	Crushi pe
Solid block of terra-cotta	523
Hollow block of terra-cotta, unfilled	186
Hollow block of terra-cotta, lightly made and unfilled	80

Tests of terra-cotta manufactured by a New York Company, wh

Terra-Cotta Piers. Tests * of terra-cotta block piers were made
same time (January, 1907, and January, 1908) that the brick piers
in Table VII were made. The tests were made on terra-cotta piers,
of which varied from 9 ft 9 in to 12 ft 7¾ in. The lateral dimensions
n 8½ by 8½ in to 17½ by 17½ in. "The piers were built and tested
, an interval of about one year separating the times of making the
e two lots of piers were built of blocks which came in different ship-
he cement used was the same brand in both years, although the lots

II. Tests of Terra-Cotta Piers, Made at the University of Illinois

unts given are average values. The table gives results of tests of piers of
ment, except for the concave-end blocks. The piers recorded in this table
½ by 12½ in by 9¾ ft.

ristics of piers	Average unit load lb per sq in	Ratio of strength of pier to strength of block, gross area	Ratio of strength of pier to strength of first of series	Crushing strength of 6-in mortar-cubes lb per sq in	Ratio of strength of pier to strength of cubes
, 1 : 3 Portland- mortar, concentri- ded............	4 300*	0.83*	Stand- ard 1.00*	....	1.26*
, 1 : 3 Portland- mortar, eccentri- ded............	3 470	0.65	0.81*	3 090	1.12
d, 1 : 3 Portland- mortar, concen- loaded...........	3 305	0.64	0.76	3 130	1.05
id, 1 : 3 Portland- mortar, eccentri- ded............	3 110	0.60	0.75	3 025	1.06
, 1 : 3 Portland- mortar, concen- loaded...........	3 050	0.59	0.71	3 370	0.88
1 : 5 Portland- mortar, concen- loaded, inferior ed blocks †.......	3 350	0.65	0.78	....	....
ith concave ends, ortland-cement	2 970	0.86	0.69	....	....

Estimated. † Blocks of good quality, but underburned.

ent. The terra-cotta block piers were generally made in sets of two.
as constructed and loaded similarly. Three of the piers were laid up
(poorly laid); the remainder were built with the usual care given to
The load was applied to the piers in different ways, although gen-
lied continuously to failure." Some piers were loaded eccentrically
and one was loaded both concentrically and *eccentrically, but the*
eccentric load was *not sufficient to cause failure.*

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etin No. 27, University of Illinois Engineering Experiment Station, Sept. 29,

bricks, these piers carrying only 31% as much as piers built with t
grade of bricks. In the case of the terra-cotta piers, the blocks w
culled out as somewhat inferior gave a pier-strength which was per
less than the piers built with superior blocks. The effect of the at
represent hurried or careless workmanship in two brick piers and in th
cotta block piers was a loss in strength of about 15% and 25% respe

"In the well-built brick piers, concentrically loaded, the ratio of st
pier to compressive strength of individual brick ranged from 31 to
in the underburned clay-brick pier the ratio was 27%. In the te
block piers, concentrically loaded, the ratio of strength of pier to that o
ual block was 74% (an incompleted test) and 83, 85 and 89% for th
The higher ratio found for the terra-cotta block piers than for brick
gests that the ability of individual pieces to resist transverse forces
ment in the strength of the completed pier; and this suggestion may
important bearing on the advantageous size of the component bloc
may be used in a compression-piece where great strength is required.

"The strength of the pier is greater than that of the mortar-cube
brick and terra-cotta block piers, except the soft-brick piers, which h
of low compressive strength. Both the strength of the individual
blocks and the strength of the mortar affect the resistance of the pier,
relative effect of the two depends upon the character of the materia
evident, however, that the better the individual piece the more impor
to have a mortar of high resisting strength.

"The results obtained in applying the loads eccentrically were found
very well with those obtained from ordinary analysis.

"The quality of workmanship in laying up such columns has an i
bearing upon the resisting strength. The work of building piers, ho
not difficult and requires only ordinary care. Full joints and an eve
are important, and the ordinary workman ought to be able to constr
of great strength. In the tests made on piers intended to represent

65 to 85 lb per cu ft, the smaller pieces weighing the most. For 18 in or larger on the face, 70 lb per cu ft will probably be a fair he tables in the manufacturers' catalogues give the various bearing-s per square foot, thicknesses of parts, sizes of blocks, etc., for por-iporous blocks for all purposes.

3. Crushing Strength of Building Stones

(1) Sandstones

low, Mass., Stone.* Reddish-brown sandstone, two blocks about cross-section and 8 in in height.

1 commenced to crack at 10 333 lb per sq in, and flew from the fragments at 13 596 lb per sq in.

2 commenced to crack at 3 012 lb per sq in and failed completely per sq in.

from Norcross Brothers' Quarries, East Longmeadow, Mass., ry Stone.* Block No. 1, 4 by 4 by 8 in high, commenced to crack nd failed at 8 812 lb per sq in.

2, 4 by 4 by 8 in high, commenced to crack at 6 500 lb and failed per sq in.

sbury Stone.* Block No. 1, 4 by 4 by 8 in high (about), commenced 12 716 lb and failed at 13 520 lb per sq in.

2, same size as No. 1, commenced to crack at 13 953 lb and failed per sq in.

ne.* Block No. 1, 6 by 6 by 6 in, commenced to crack at 12 590 lb 12 619 lb per sq in.

2, same size as No. 1, commenced to crack at 12 185 lb and failed per sq in.

one from the Shaler & Hall Quarry Company, Portland, Conn.† of the tests are as follows:

Table IX. Crushing Strength of Brown Sandstone

imensions		Sectional area	First crack	Ultimate strength	Classification
Compressed, surface in		sq in	lb	lb per sq in	
2.50	2.45	6.13	84 800	13 980	1st quality
2.48	2.47	6.13	81 700	13 330	1st quality
3.00	2.95	8.85	123 200	13 920	2d quality
2.98	2.97	8.85	122 000	15 020	3d quality
2.55	2.53	6.45	63 850	9 900	Bridge
2.48	2.52	6.25	58 340	9 330	Bridge

one from the Middlesex Quarry Company, Portland, Conn.‡ Four al blocks, about 1½ in square. Pressure per square inch at time of 1, 10 928 lb; No. 2, 10 322 lb; No. 3, 8 252 lb and No. 4, 6 322 lb.

sts were made with the United States testing-machines *at Watertown* s.

sts made *by Colt's Patent Fire-arms Manufacturing Company.*

sts were made with the United States testing-machines at Watertown

Gray Sandstone † from Fort Collins, Col. (laminated and simila to the St. Vrain stone). Crushing strength on bed, 11 700 lb per sq 140 lb per cu ft. One ton of this stone measures just a perch in the

(2) Granite

Red Granite † from Platte Cañon, Col. Crushing strength per s 14 600 lb; weight per cubic foot, 164 lb.

(3) Lava Stones

Lava Stone from the Kerr Quarries, near Salida, Col. Four cubi The results of the tests are as follows:

Table X. Crushing Strength of Lava Stone

Dimensions			Sectional area	First crack	Ultimate
Height in	Compressed surface in		sq in	lb	lb
4.00	4.00	4.00	16.00	165 900	165 000
4.00	4.00	4.00	16.00	174 100	174 100
2.00	2.00	1.99	3.98	36 400	37 100
1.99	1.99	1.99	3.96	38 200	38 200

Lava Stone,‡ Curry's Quarry, Douglas County, Col. Crushin 10 675 lb per sq in; weight, 119 lb per cu ft. Experience has show stone is not suitable for piers, or where any great strength is req cracks very easily.

(4) Marble and Limestone

White marble quarried at Sutherland Falls, Vt. Two cubical b

(5) Bricks and Various Stones

gives the crushing strength of various kinds of bricks and building ressure being normal to the plane of the bed.

Table XI. Crushing Strength of Brick and Stone *
Pressure at right-angles to bed

Kind of brick or stone	Crushing strength, lb per sq in
n, Massachusetts	10 000
1, St. Louis, Mo.	6 417
1, Washington, D. C.	7 370
Illinois	6 000 to 13 000
s Island, Me.	14 875
inal Haven, Me.	13 000 to 18 000
, R. I.	15 000
t and Quincy, Mass.	17 750
Conn.	22 600
sland, N. Y.	22 250
Cloud, Minn.	28 000
n, Col	13 000
tte Cañon, Col.	14 600
s:	
lls, N. Y.	11 475
	12 775
Ind.	6 000 to 10 000
nd.	8 625
g. Minn.	23 000
r, Minn.	10 750
s:	
ter, N. B. (brown)	9 150
Point, N. B. (fine grain, dark brown)	7 700
icut brown stone,† (on bed)	7 000 to 13 000
adow, Mass. (reddish brown)	7 000 to 14 000
adow, Mass. (average, for good quality)	12 000
lls, N. Y.	9 850
N. Y.	17 000
, N. Y. (red)	18 000 to 42 000
d, Ohio.	6 800
mherst, Ohio	6 212
hio	8 000 to 10 000
stown, Pa.	12 810
Lac, Minn.	8 750
Lac, Wis.	6 237
, Col. (light red)	6 000 to 11 000
, Col. (hard laminated)	11 505
s.	22 900
, Vt.	10 746
nery Co., Pa.	10 000
Cal.	17 783
	12 156
iver, N. Y.	13 425

complete tables *of the strength, weight and composition of building stones, tables, etc. by Professor Thomas Nolan in Kidder's Building Construction dence, Part I, Masons' Work.*

hould not be set on edge.

2.72; crushing strength, 15 000; shearing strength, 2 000; modulus o
1 500; modulus of elasticity, 7 000 000. Limestone: weight, 170
gravity, 2.72; crushing strength, 6 000; shearing strength, 1 000;
rupture, 1 200; modulus of elasticity, 7 000 000. Marble: weight,
cific gravity, 2.72; crushing strength, 10 000; shearing strength, 1
ulus of rupture, 1 400; modulus of elasticity, 8 000 000. Slate: w
specific gravity, 2.80; crushing strength, 15 000; modulus of ruptu
modulus of elasticity, 14 000 000. Trap-Rock: weight, 185; specif
2.96; crushing strength, 20 000.

The following average relative values are given by A. I. Frye.‡
the results of tests made on small cubes of the materials. Sandston
ing strength, 9 000 lb per sq in; Granite and Gneiss: crushing
17 733 lb per sq in. Limestones and Marbles: crushing strength,
per sq in. Slate: crushing strength, 10 000; ultimate tensional
3 000; modulus of rupture, 5 000 lb per sq in.

When stones are not tested, Frye recommends the following aver
for ultimate strengths to be used in determining the safe stresses. S
crushing strength, 5 000; ultimate tensional strength, 150; modulus o
1 200 lb per sq in. Granite and Gneiss: crushing strength, 12 0
ulus of rupture, 1 600 lb per sq in. Limestones and Marbles:
strength, 8 000; ultimate tensional strength, 800; modulus of ruptur
per sq in.

The following working unit stresses in pounds per square inch for s
or single blocks of stone are recommended by W. J. Douglass.§ S
compression, 700; tension (direct and flexural), 75; shear, 150.
Syenite and Gneiss: compression for hard, 1 500; for medium, 1
soft, 1 000; tension (direct and flexural), 150; shear, 200. Limesto
pression for hard, 1 000; for medium, 800; for soft, 700; tension (
flexural), 125; shear, 150. Marble: compression for hard, 900; for
tension (direct and flexural), 125; shear, 150. Bluestone Flagging:
sion, 1 500; tension (direct and flexural), 200.

4. Compressive Strength of Mortars and Concretes

pressive Strength of Natural-Cement Mortar. The crushing
natural-cement mortar, neat, averaged, for 7 days, 2 010; for
; for 3 months, 3 646; and for 6 months, 5 052 lb per sq in. When
parts of standard quartz sand, the mortar averaged in crushing
7 days, 940; for 28 days, 1 390; for 3 months, 1 730; and for
012 lb per sq in. For 2 years, an additional increase of 18% and
assumed for the neat and sanded mortars, respectively, of natural

pressive Strength of Portland-Cement Mortar. The crushing
-Portland-cement mortar, neat, averaged, for 7 days, 5 915; for
41; for 3 months, 7 347; and for 6 months, 9 760 lb per sq in.
with 3 parts of standard quartz sand, the mortar averaged, in
ngth, for 7 days, 941; for 28 days, 1 290; for 3 months, 1 490;
nths, 1 529 lb per sq in. When mixed with 3 parts of Ottawa sand,
veraged, in crushing strength, for 7 days, 1 199; for 28 days, 1 796;
s, 1 887; and for 6 months, 2 181 lb per sq in. For 2 years, an
crease of about 16% and 18% may be assumed for the neat and
ars, respectively, of Portland cement.

of Compressive to Tensile Strength of Mortars. While it
ed as a very general guide that the compressive strength of hy-
nt mortars is from six to ten times the tensile strength, these ratios
and cannot be used as a reliable basis for calculations. The tensile
Portland-cement mortars, under normal conditions, increases
g the first few days, the rate of change gradually falling off. In
ensile strength is generally from one-half to two-thirds of the ulti-
h, which is practically reached in 2 or 3 months. The compressive
vever, continues to increase with age and the rate of increase varies
a somewhat different law.

pressive Strength of Concrete. There are many reasons for
s in the values of the compressive strength of concrete and the
tors are (1) the quality of the cement, (2) the size and character
gates, (3) the quantity of the cement to a unit volume of the con-
e manner of mixing, (5) the density of the mixture, (6) the condi-
which it seasons, and (7) its age; and of these various conditions
determination of the compressive strength the most important
the proportions of the different ingredients of the mixture and its
gh tables of average values of ultimate crushing strengths of con-
blished and are of general value, they may be misleading unless
ith caution. In important operations it is advisable to have the
ed and to adjust by trial the character and proportions of the in-
til the required strength is obtained.

Specimen for Compression-Tests. For compression-tests of
general, 4 to 12-in cubes of the mixture have been the standard
t-specimens; but since the advent of reinforced-concrete construc-
growth of the importance of determining the elastic properties of
has been found that a cylindrical test-specimen gives more definite
a cube. A common shape of such cylinder is one in which the
out three times the diameter, and the cylinders are not less than
It is found that the compressive strengths of these *cylinders of*
from 10 to 15% less *than those of the cubes, but for cylinders of*

---

pression-tests made by W. P. Taylor on cylindrical specimens 1 in in height,
diameter and 1 sq in in cross-section.

still greater slenderness the compressive strengths remain about cons heights up to about seven diameters.

Compression-Tests on Concrete Cubes. From some tests made for the Boston Elevated Railway Company at the Watertown Arsenal, cubes of concrete made with five brands of Portland cement, coars sand and broken stone up to 2½-in size, having 49.5% voids, the f average values of the compressive strengths were obtained:

Table XII. Compression-Tests on Concrete Cubes

Mixtures	7 days lb per sq in	1 month lb per sq in	3 months lb per sq in	6 months lb per sq in
1 : 2 : 4	1 560	2 400	2 900	3 820
1 : 3 : 6	1 310	2 160	2 520	3 090

Compression-Tests on Concrete-Cylinders. For cylindrical te mens of concrete, made under reasonably good conditions as to cha materials and care in mixing, an average compressive strength of abo lb per sq in is usually developed in a 1 : 2 : 4 Portland-cement concret 1 to 2 months; and of about 1 600 lb per sq in in a 1 : 3 : 6 mixture the conditions are unusually favorable somewhat higher values than t obtained, but when the materials and workmanship are poor the ultim pressive stresses are lower.

Increase in Compressive Strength of Portland-Cement Concr regard to the increase of compressive strength of Portland-cement with age, tests show that the ultimate compressive strength is nearly re 60 days, at which time the strength varies from 80 to 90% of its value in time.

Ultimate Strengths of Natural-Cement Concrete. For natura concrete, the ultimate compressive, tensile and shearing strengths and ulus of rupture may be taken at about one-half the corresponding v Portland-cement concrete, unless natural cements of known and teste are employed.

Strength of Unreinforced Concrete Columns. Short concrete of lengths up to 10 or 15 diameters, develop a crushing strength of fr

Table XIII. Compression-Tests on Unreinforced Concrete Colum

Kind of concrete	Average age days	Average ultimate compressive stress lb per sq in
1 : 1 : 2	60	3 600*
1 : 1½ : 3	60	2 270
1 : 2 : 4	60	1 600
1 : 2½ : 5	60	1 200
1 : 3 : 6	60	935
1 : 3½ : 7	60	745
1 : 4 : 8	60	600

---

* This value was estimated as it was beyond the range of the testi

ula

S_a = the ratio of sand to cement
S_t = the ratio of stone to cement

In the 1 : 3 : 6 mixture, $S_a = 3$ and $S_t = 6$

Strength of Concrete Affected by Area of Bearing Surface. ol states † that if a load is applied over the central part, only, surface of a concrete test-specimen in compression, the unit load r than if it is applied over the entire surface; and this is due to the outer parts tend to assist the inner part to resist the stress. This tests made on some of the 12-in concrete cubes used in the tests Boston Elevated Railway Company and referred to in the preced- s. Thirty-six of these concrete cubes were crushed by applying the entire upper bearing-surface of 144 sq in and an equal number crete cubes were then crushed by applying the stress over a smaller in, or 100 sq in. After this, the cubes of a third set were crushed ation of the stress over the still smaller area, 8 by 8¼ in, or 66 ests of the second set gave unit crushing strengths 12% higher than those of the third set unit crushing strengths 28% higher than

tress for Bearing on Concrete. "When compression is applied f concrete of at least twice the loaded area, a stress of 32.5% of ve strength may be allowed."‡

Stress for Axial Compression on Concrete. "For concentric n a plain concrete column or pier, the length of which does not meters, 22.5% of the compressive strength may be allowed." * gth of reinforced-concrete columns, see Chapter XXIV, page 945.)

nded Ultimate Compressive Strengths of Portland-Cement Table XIV, of ultimate compressive strengths of concrete of ures gives the values recommended by the American Society for erials, even though occasional tests show higher results. The are recommended as the maximum ultimate unit compressive should be used in design and on which the permissible working

	lb per sq in	lb per sq in	lb per sq in	lb per sq in
Granite, trap-rock..........	3 300	2 800	2 200	1 800
Gravel, hard limestone and hard sandstone...........	3 000	2 500	2 000	1 600
Soft limestone and sandstone......................	2 200	1 800	1 500	1 200
Cinders....................	800	700	600	500

Effect of Consistency on the Crushing Strength of Concrete.
that is mixed fairly dry and tamped until the moisture is brought to t
develops a somewhat greater compressive strength than concrete r
more water. From a large number of tests * average compressive s
wet, plastic and dry concretes were determined. The age of the co
1 year and 8 months, and five brands of cements were used. The
pressive strengths were, for the wet concrete, 2 130; for the pla
and for the dry, 2 350 lb per sq in.

In another series of tests † greater differences appeared. At th
month the mean compressive strengths in pounds per square inch w
wet concrete: granite, 3 155; gravel, 2 300; limestone, 4 195. For t
concrete: granite, 4 090; gravel, 3 545; limestone, 2 975. For the
crete: granite, 4 520; gravel, 4 610; limestone, 4 365. At the end o
the values for the granite aggregates were, for the wet concrete, 4 7
medium, 4 990; and for the damp, 5 445.

Effect of Size of Stone on the Compressive Strength of Con
may be stated, generally, that the use of stones of a maximum size
with convenience generally results in a maximum compressive stren
concrete. Stones of the larger sizes are generally more uniformly g
the smaller stones, and consequently grade better with the sand and g
strength. From tests ‡ made by W. B. Fuller, the average compressive
at 140 days, of 1 : 9 concrete, were, for maximum size of stone ½ i
per sq in; for 1-in stone, 1 150 lb per sq in; and for 2½ in stone

of the Strength of the Aggregate on the Compressive Strength retes. The compressive strength of trap-rocks, granites and most s is relatively so great that it cannot reduce the strength of the conlf. Some sandstones, however, have a much lower average compressive and if they are friable and soft may lower relatively the final strength ncrete. A concrete of low strength results from using cinders for the

CHAPTER VI

FORCES AND MOMENTS

By

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1. Composition and Resolution of Forces

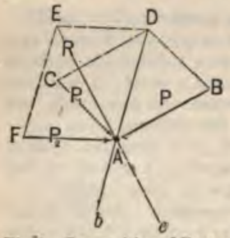


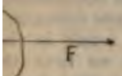
Fig. 1. Composition of Forces

Composition and Resolution of Forces. Imagine a round ball pl plane, frictionless surface at A (Fig. 1), the surface being perfectly leve the ball has no tendency to move until s is applied to it. If, now, the force, P, i to the ball in the direction indicated by t the ball will move in that direction. If, i one force only, two forces, P and P_1, are a the ball, it will not move in the direction of the forces, but will move in the directi RESULTANT of these forces, or in the dire If the magnitudes of the forces P and indicated by the lengths of the lines, th complete the parallelogram $ABDC$, the DA represents the direction and magnit single force which has the same effect on as that resulting from the two forces P and P_1. If, in addition to forces P and P_1, the third force, P_2, is applied, the ball will mov direction of the resultant of all three forces, and this resultant is obt completing the parallelogram $ADEF$, of which the resultant DA and force P_2 are two adjacent sides. The diagonal R of this second parallelogram is the resultant of all three forces, and the ball will move in the direction Ae. In the same way the resultant of any number of forces may be found. Again, suppose a ball, whose weight is indicated by the length of the line W (Fig. 2), is suspended by two inclined cords. What are the magnitudes of the pulls or stresses which are developed in the cords and which keep the ball suspended at the point A? This is the converse of the last case. Instead of finding the diagonal or the resultant, the diagonal, which is the line W, is given, and the sides of the parallelogram are to be found. To find these the lines representing the directions of P and P_1 are prolonged and from B lines parallel to them are drawn to complete the parallelogram. Then CA is the required magnitude of the stress in cord P, and BC of that in cord P_1. Thus one f have the same effect as many, or many the same effect as one.

Fig. 2.
tion o

Forces Represented by Straight Lines. In considering the action it is convenient to represent them graphically by straight lines wit heads, as in Fig. 3. The length of the line, if drawn to a scale of poun sents the MAGNITUDE OF THE FORCE in pounds; the position of the lin

ACTION; the arrow-head indicates its SENSE or the direction in which nd the point A its POINT OF APPLICATION. Thus the magnitude, direction and point of application are indicated and the force is completely represented.



rce Represented
raight Line

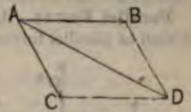


Fig. 4. Parallelogram of Forces

Parallelogram of Forces. If two forces applied at one point are represented in magnitude and direction by two nes inclined to each other, their resultant is the diagonal of the GRAM formed on those lines. Thus, if the lines AB and AC (Fig. 4) two forces acting at a point A, to find the force which will have the t as the two forces, the parallelogram completed and the diagonal AD drawn. represents the RESULTANT of the two hen the two given forces act at right-ach other, the magnitude of the resultant o the square root of the sum of the the magnitudes of the other two forces.

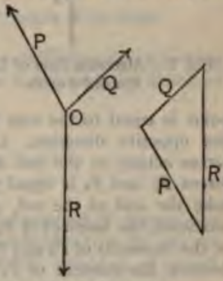


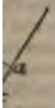
Fig. 5. Triangle of Forces

e of Forces. If three forces acting at represented in magnitude and direction s of a TRIANGLE taken in order, they are um. Let P, Q and R (Fig. 5) represent s acting at the point O. If a triangle wn, like that shown at the right in Fig. sides respectively parallel to the direc-e forces and taken in the same order, are in equilibrium. If such a triangle cannot be drawn, the forces equilibrium.

lygon of Forces. If any number of forces acting at a point can nted in magnitude and direction by the sides of a POLYGON taken hey are in equilibrium. This follows directly from the preceding

2. Moments of Forces

ts. In considering the stability of structures and the strength of we are often obliged to take into consideration the moments of acting on a structure or on some part of a structure; and a knowledge of the general PRINCIPLES OF MOMENTS is essential to the proper understanding of these subjects. When we speak of the MOMENT OF A FORCE, we must have in mind some fixed point or line with respect to which the moment is taken. The moment of a force with respect to any given point, or CENTER OF MOMENTS, is the product of the magnitude of the force and the perpendicular distance from the point to the LINE OF ACTION of the force; or, in other words, the moment of a force is the product of the magnitude of the force by ith which it acts. Thus if we have the force F *(Fig. 6), and wish to* its moment with *respect to the point P, we determine the per-distance Pa, between the point and the line of action of the force, it by the magnitude of the force. For example, if the magnitude*



foment
rce

of the force F is 500 lb and the distance Pa is 2 in, the moment o with respect to the point P is 500 lb × 2 in = 1 000 in-lb.*

Parallel Forces. If any body is in a state of rest or equilibrium action of parallel forces, the sum of the forces acting in one direction sum of the forces acting in th direction. Thus if we have t forces P^1, P^2, P^3 and P^4 acting AB (Fig. 7), in a direction oppo of the forces P_1, P_2 and P_3, then is in equilibrium, the sum of the P^2, P^3 and P^4 must equal the s forces P_1, P_2 and P_3.

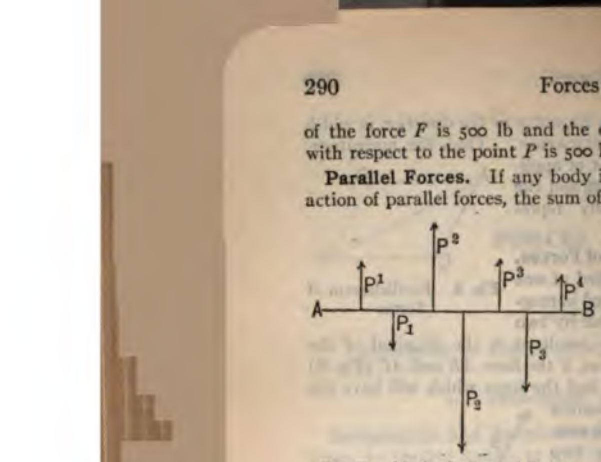


Fig. 7. Algebraic Sum of Unlike Parallel Forces

Parallel Forces Opposite in If any number of parallel forc acting in the same direction, act if the body is in equilibrium, the moments of the forces tending t body in one direction about point is equal to the sum of the moments of the forces tending to the opposite direction. Let F_1, F_2 and F_3 (Fig. 8) represent thr forces acting on the rod AB. If the rod is in equilibrium, the s forces F_2 and F_3 is equal to F_1. Also, if we take the end of the rod, A, for the center of moments, the moment of F_1 is equal to the sum of the moments of F_2 and F_3 about that point, because the moment of F_1 measures the tendency to turn the rod CLOCKWISE, and the sum of the moments of F_2 and F_3 measure the tendency to turn the rod CONTRA-CLOCKWISE, and there is no more tendency to turn the rod one way than the other. For example, let the magnitude of forces F_2, F_3 each be represented by 5 force-units, the distance Aa by 2 length-units and the distance AB by 4 length-units. The magnitude of the force F_1 must equal the sum of the magnitudes of the forces F_2 and F_3, or 10 force-units, and i with respect to any point in the plane of the forces must equa of the moments of F_2 and F_3 with respect to the same point. If we the center of moments, the moment of $F_3 = 5 \times 2 = 10$, and of $F_2 =$ Their sum equals 30; hence the moment of F_1 must be 30. Dividi ment 30 by the force $F_1 = 10$ force-units, we have for the arm, 3 lengt the force F_1 must act at a distance of 3 units from A to keep the rod rium. If we take b as the center of moments, the force F_1 has no m the length of its lever-arm is zero; and, for equilibrium, the moment b must equal the moment of F_3 about the same point; or, as in th magnitudes of the forces F_2 and F_3 are equal, they must both be app same distance from b, showing that b must be half-way between a an demonstrated before.

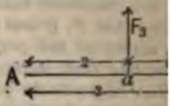


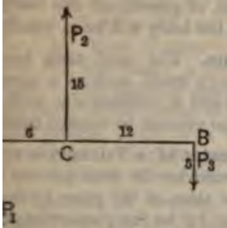
Fig. 8. Algebraic S ments of Unlike Pa

Three Parallel Forces. THE PRINCIPLE OF THE LEVER. This p based upon the two preceding propositions and is of great importanc

---

[*] The expressions POUND-FEET and POUND-INCHES are often given to th to distinguish them from FOOT-POUNDS and INCH-POUNDS, by which WORK are measured.

If a body is in equilibrium under the action of three parallel forces e same plane, each force is proportional to the normal distance be- ther two. Thus, if, as in Figs. 9, 10 and 11, three forces, P_1, P_2 and



9. Principle of the Lever

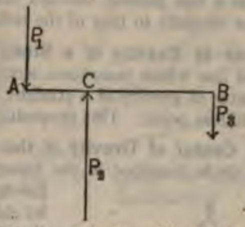


Fig. 10. Principle of the Lever

the rod AB, in order that it may be in equilibrium, the following ust obtain between the magnitudes of the forces and the distances eir points of application;

$$\frac{P_1}{CB} : \frac{P_2}{AB} : \frac{P_3}{AC}$$

$$P_1 : P_2 : P_3 :: CB : AB : AC$$

case of the COMMON LEVER and shows the method of determining t a given lever will raise. The proportion is also true for any ar- of the forces (as shown in Figs. 9, 10 and 11), provided, of course, are lettered in the order shown in the

ple, let the distance AC be 6 in and the 3 be 12 in. If a weight of 500 lb is the point B, how much will it raise at nd and what support will be required at

the rule just given, we have the pro-

$_1 :: AC : CB$ or $500 : P_1 :: 6 : 12$

1 000 lb; or 500 lb applied at B will resting on or suspended at A. The force at C must, by the principles of PARALLEL FORCES IN , be equal to the sum of the forces P_1 and P_3, or 1 500 lb in

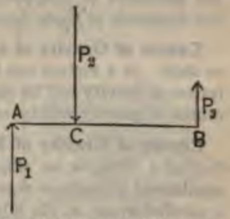


Fig. 11. Principle of the Lever

3. Center of Gravity

Principles. The LINES OF ACTION of the force of gravity converge center of the earth; but the distance of the center of the earth from which we have occasion to consider, compared with the size of those o great, that we may consider the lines of action of the forces as he number of the forces of gravity acting upon a body may be con- qual to the number of particles composing the body. *The* CENTER OF a body may be defined *as the point through which the resultant of forces of gravity, acting upon the body, passes for every position If a body is supported at its center of gravity and turned about*

The center of gravity of a straight line or rod of uniform size and
at its middle point. This proposition is too evident to require de

The Center of Gravity of the Perimeter of a Triangle is
of the circle inscribed in the triangle formed by the lines joining
points of the sides of the given tria
let *ABC* (Fig. 12) be the given triang
the center of gravity of its perime
middle points, *D*, *E* and *F*, and conn
straight lines. The center of the ci
in the triangle formed by these lines
center of gravity sought.

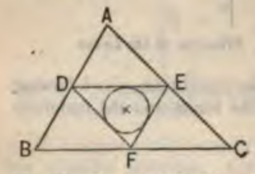


Fig. 12. Center of Gravity of Perimeter of Triangle

Center of Gravity of Symmet
The center of gravity of a line wh
metrical with reference to a point
point. Thus the center of gravity
cumference of a circle or of an ellipse is at the geometrical center of
The center of gravity of the perimeter of an equilateral triang
regular polygon, is at the center of the inscribed circle. The center
the perimeter of a square, rectangle, or parallelogram is at the in
the diagonals of those figures.

Center of Gravity of a Surface. A SURFACE here means a ve
or shell. If a surface can be divided by a line into two symmetrical
center of gravity will be on that line; if it can thus be divided by t
center of gravity will be at their intersection.

Center of Gravity of Regular Figures. The center of gravity
face of a circle or an ellipse is at the geometrical center of the fi
equilateral triangle or regular polygon, at the center of the inscribe
a parallelogram, at the intersection of the diagonals; of the surface

, will form a triangle whose center of gravity is that of the quad- Thus, let Fig. 13 be a quadri- ose center of gravity is to be aw the diagonals AD and BC, l lay off $AF = DE$, and from B $= CE$. From E draw a line to of FH, and from F a line to the EH. The point of intersection o lines is the center of gravity adrilateral. This is a method used for finding the centers of the voussoirs of an arch.

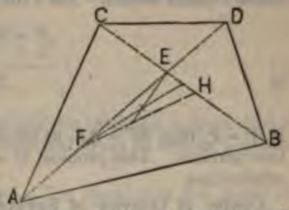
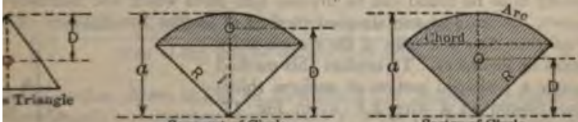


Fig. 13. Center of Gravity of Quadrilateral

of Centers of Gravity. Let a rawn from the vertex of a figure dle point of the base, and D the distance from the vertex to the center of the figure. Then (Fig. 14):

isosceles triangle	$D = \frac{2}{3} a$
egment of a circle, vertex at center of circle	$D = \frac{\text{chord}^3}{12 \times \text{area}}$
sector of a circle, vertex at center of circle	$D = R \times \frac{2 \times \text{chord}}{3 \times \text{arc}}$
semicircle, vertex at center of circle	$D = \frac{4R}{3\pi} = 0.4244 R$
quadrant of a circle	$D = \frac{3}{5} R$
semiellipse, vertex at center of circle	$D = 0.4244 a$
parabola, vertex at intersection of axis h curve	$D = \frac{3}{5} a$
cone or pyramid	$D = \frac{3}{4} a$



Triangle

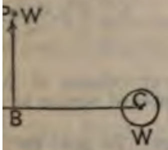
Segment of Circle

Sector of Circle

Fig. 14. Center of Gravity of Triangle, Segment and Sector

um of a cone or pyramid, let h = the height of the complete cone or pyramid, h_1 = the height of the frustum, and let the vertex be at the apex of the complete cone or pyramid; then,

$$D = \frac{3(h^4 - h_1^4)}{4(h^3 - h_1^3)}$$



Center of Gravity of Heavy Particles

Center of Gravity of Two Heavy Particles. Let P be the weight of a particle at A (Fig. 15), and W that of a particle at C. The center of gravity is at some point, B, *on the line joining A* The point B must be so *situated that if the two particles were held by a stiff wire and supported at B by a force equal in magnitude to f P and W they would be in equilibrium.* The problem then is solved by the PRINCIPLE OF THE LEVER, and we have the pro Three Parallel Forces. The Principle of the Lever),

$$P + W : P :: AC : BC$$

or

$$BC = \frac{P \times AC}{P + W}$$

If $W = P$, then $BC = AB$, or the center of gravity will be half-way two particles. This problem is of great importance and has ma applications.

Center of Gravity of Several Heavy Particles. Let W_1, and W_5 (Fig. 16) be the weights of the particles. Join W_1 and W_2 b line and find their center of gravity A preceding problem. Join A with W_3 a center of gravity B, which will be th gravity of the three weights W_1, W_2, W in the same way with each weight. The of gravity found will be the center of gr the particles. In both of these cases the the particles are supposed to be horizon else the horizontal projections of the st which join the points.

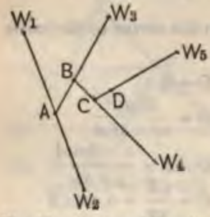


Fig. 16. Center of Gravity of Several Heavy Particles

Center of Gravity of Compound Found by Moments. To determine th of a beam having an unsymmetrical cros is first necessary to determine the distance of the center of gravity of from the upper or lower surface of the beam. Various other con also, involve finding the center of gravity of an irregular figure, problem is one of practical importance. If the figure of which th gravity is to be found can be divided into parts which are themsel figures, the readiest and simplest method of finding the distance of of gravity from one edge of the section is by means of MOMENTS. To explain this method assume a T-shaped section of uniform thickness, hinged on a wire XX, as in Fig. 17. The T section is made up of two rectangles, one forming the flange, the other the web. The center of gravity of each rectangle is at its own center of figure and may be readily found. If the T section is placed horizontally, as in the figure, the axis XX being fixed, it will immediately, by the force of gravity, revolve about the axis until it becomes vertical, and the sum of the moments of the forces causing the revolution is $A' \times d' + A'' \times d''$, A' representing the weight of the web and A'' the weight of the flange. To hold the in a horizontal position, there must be a moment of some force a opposite, or upward, vertical direction and just equal to the s two moments causing revolution downwards. If the force A, of th tending to cause revolution upward, is equal to the weight of the en tion, it must be applied at the center of gravity of the entire figure moment just equal to the sum of the moments of the two down

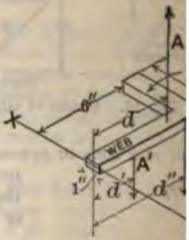


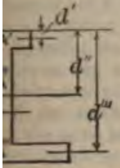
Fig. 17. Center of Compound Sections by

oment of A is $A \times d$, therefore d is the distance from the end of the web, he axis XX, to the center of gravity of the entire figure. Therefore, $d = A' \times d' + A'' \times d''$,

$$d = \frac{A' \times d' + A'' \times d''}{A} \qquad (1)$$

ight of any homogeneous material of uniform thickness is proportional ea, A, A' and A'' may be used to represent areas as well as weights. g formula (1) as a rule, we have:

of Gravity of Compound Figures. The distance of the center of f a compound figure from any line of reference is equal to the sum of ıcts, obtained by multiplying the area of each of the simple parts into e compound figure is divided by the distances of its center of gravity from the line of reference, divided by the area of the entire figure. This rule applies to any compound figure.



Center of Grav-
Tees, Angles,
ls, etc.

Example I. Assume that the ⊤ section shown in Fig. 17 has the dimensions indicated. Then A' equals 6, A'' equals 8, and A equals 14 sq in; and d' equals 3 and d'' equals 6½ in. The sum of the products of A' by d' and ' is 18 + 52 or 70 sq in × in, and this divided by 14 sq in, the area tire figure, gives 5 in for the distance d. The distance d of the center from the top of the webs, in each of the figures shown in Fig. 20, by the following formula:

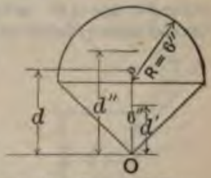


Fig. 19. Center of Gravity of Irregular I Section

$$d = \frac{\text{area of the web or webs} \times d'/2 + \text{area of flange} \times d''}{\text{area of the web or webs} + \text{area of flange}} \qquad (2)$$

tion like that shown in Fig. 18, in which A', A'' and A''' represent the he respective rectangles, the distance d of the center of gravity from ay be found by the formula

$$d = \frac{A' \times d' + A'' \times d'' + A''' \times d'''}{A' + A'' + A'''} \qquad (3)$$

e II. To show the application of the rule for finding the center of f compound figures, take the one shown in Fig. 19. The distance d

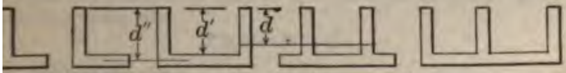


Fig. 20. Center of Gravity of Irregular Figures

ter of gravity of the entire figure from the vertex O is *found as follows:* of the triangle is 36 sq in *and of the semicircle 56.5 sq in. From the Centers of Gravity (page 293) the distance of the center of gravity of triangle from the vertex is two-thirds its height,* which gives 4 in as the value for d'. The center of gravity for a semicircle is $0.4244R$ base, so that d'' equals 8.54 in. Then,

$$d = \frac{36 \times 4 + 56.5 \times 8.54}{36 + 56.5} = 6.77 \text{ in}$$

This method of finding the center of gravity is similar to that e Chapter IX for finding the supporting forces or reactions. In the l however, the problem is to find the balancing forces instead of the leve

CHAPTER VII

STABILITY OF PIERS AND BUTTRESSES

BY
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nical Principles. A pier or buttress may be considered STABLE when
acting upon it do not cause it to ROTATE or TIP OVER nor cause any
nasonry to SLIDE on its bed; some parts, however, of the masonry may
ED. When a pier sustains a vertical load only, it might be con-
ABLE, but it might not have sufficient STRENGTH. It is only when the
es a THRUST, as from a rafter or an arch, that its stability must be con-
In order that there may be no rotation,
NT OF THE THRUST (Chapter VI) against
bout any point in its outside edge must
d the MOMENT OF THE WEIGHT of the
the same point.

trate let us consider the pier shown in
et us suppose that this pier receives the
rafter which exerts a THRUST T in the
AB. The tendency of this thrust is to
pier to rotate about the outer edge b_1,
OMENT OF THE THRUST about this point,
the measure of this tendency to rotate,
$a'b_1$ being the lever-arm of the moment.
BLE EQUILIBRIUM, only, the MOMENT OF
HT of the pier about the same edge must just equal $T \times a'b_1$. The
force representing the weight of the pier acts vertically through its
ravity which in this case is equidistant from its sides; and its lever-
or one-half its thickness.

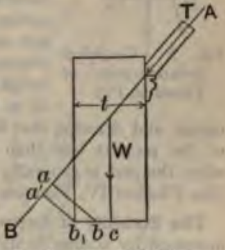


Fig. 1. Pier with Thrust

or equilibrium of moments, we must have the equation

$$T \times a'b_1 = W \times b_1c$$

his condition the least additional thrust, or the crushing of the outer
cause the pier to rotate; hence, for safety, we must use some FACTOR
. This is sometimes done by making the moment of the weight equal
the thrust when referred to a point in the bottom of the pier, a cer-
ce in from the outer edge. This distance for piers or buttresses should
than one-fourth the thickness of the pier.

nting this point in the figure by b, we have the necessary equation for
ability of the pier

$$T \times ab = W \times t/4$$

width of the pier.

not from this equation determine the dimensions of a pier to resist a
st. because we have the distance ab, t and W, all unknown quantities.
must first assume a tentative size for the pier, find *the length of the*
d see if the MOMENT OF *THE WEIGHT of the pier is equal to the* MO-
HE *THRUST. If it is not we must assume another size for the pier.*
fact the steps of the problem usually present themselves in the

if a given pier or buttress is capable of resisting a given thrust, the pro be solved GRAPHICALLY in the following mann ABCD (Fig. 2) represent a pier which sustains thrust T at B. To determine whether the pier w sustain this thrust, we proceed as follows:

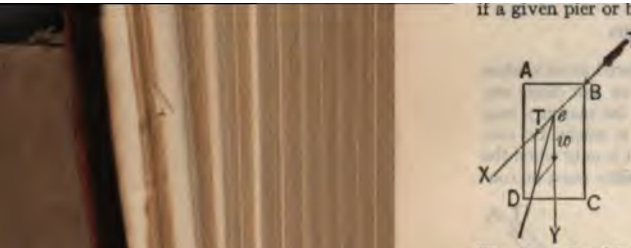


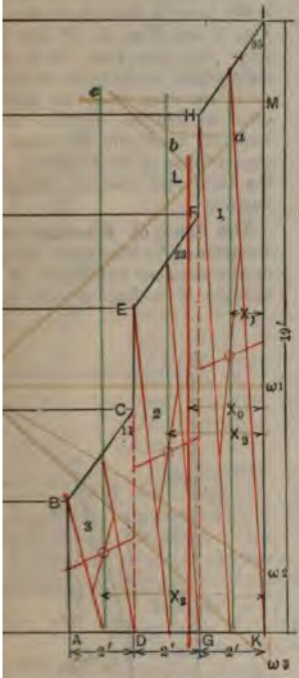
Fig. 2. Graphical Determination of Thrust on Pier

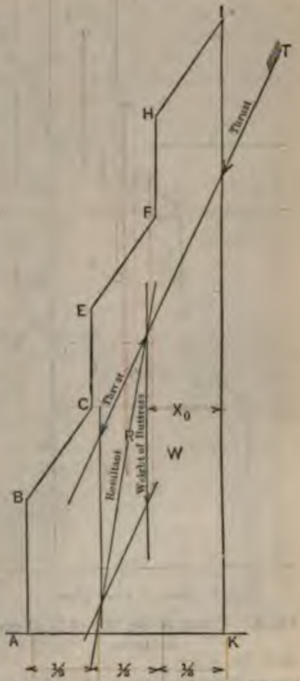
Draw the indefinite line BX in the directio thrust. Through the center of gravity of the pi in this case is midway between AD and BC, dra tical line intersecting the line of the thrust at force may be considered to act anywhere along it action, we may consider that the thrust and th act at the point e. The resultant of these two obtained by laying off the thrust T from e on eX, weight of the pier W, from e on eY, both to the sa of so many pounds to the inch, completing the ogram and drawing the diagonal. If this diagonal, prolonged, cuts of the pier at less than one-third the width of the base from t edge, the pier is generally considered unstable and its dimensions are (See Chapter IV, Theorem of the Middle Third.)

The Stability of Buttress with Offsets. THE STABILITY OF A PIER increased by adding to its weight by placing some heavy material on example, or by increasing its width at the base by means of OFFSET Fig. 3. Figs. 3 and 4 show the method of determining the stability o tress with offsets. The first step is to find the vertical line passing thr center of gravity of the whole pier. This is best done by dividing the into quadrilaterals, as $ABCD$, $DEFG$ and $GHIK$ (Fig. 3), finding the gravity of each quadrilateral by either the method of diagonals or tri explained in Chapter VI, and then measuring the perpendicular dista X_2, X_3 from the different centers of gravity to the line KI. (See, also, VIII, page 313).

Multiply the area of each quadrilateral by the distance of its center ity from the line KI and add together the areas and the products. Di sum of the products by the sum of the areas and the result will be the of the moments of the areas is 155.55, and dividing this by the total ve 2.25 as the distance X_0. Measuring this to the scale of the draw- *I*, we have a point through which the vertical line passing through the ravity must pass.

, passing through the center of gravity of the buttress, can be found LY, also, by the method of the EQUILIBRIUM POLYGON (Fig. 3). (See,





3. Buttress with Offsets

Fig. 4. Resultant Thrust on Buttress with Offsets

ter VIII, Fig. 12, etc.) In order to do this, lay off at any convenient nning at some convenient point *M*, *Mw* 1, *w* 1 *w* 2, and *w* 2 *w* 3, the e various quadrilaterals composing the buttress. Through the center of each quadrilateral draw a vertical (green) line. Draw the lines *w* 3 *O*, intersecting at some conveniently chosen POLE-POINT, *O*. 1 and *Ow* 2. Through *a*, where *MO* intersects the vertical ne drawn through the center of gravity of the first quadrilateral, arallel to *Ow* 1, and through *b*, where *ab* intersects the (green) line ne center of gravity of the second quadrilateral, draw *bc* parallel to nally draw *cL* parallel to *Ow* 3. *Where this line intersects MO* e the point *through which the (heavy red) line, passing through the ravity of the buttress taken as a whole, should be drawn.* The distance

X_0, measured from IK, should then be 2.25 ft or very nearly this, slight errors of drawing, and the same as that found by MOMENTS. the same method of determini tion of the center of gravity similar to the one illustrated

After this line is found, of determining the stability is the same as that given for Fig. 2 and Fig. 4. If the more than one foot thick, right-angles to the plane of cubic contents must be de order to find its weight. however, to divide the tota the thickness of the buttress. the thrust per foot of thick buttress.

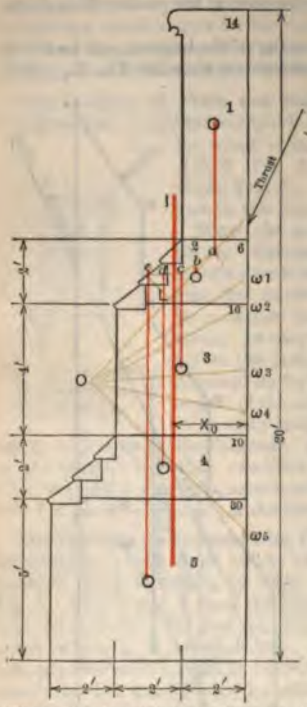


Fig. 5. Center of Gravity of Wall and Buttress

The Line of Pressure Resistance.* The LINE OF or the LINE OF PRESSURE o buttress is a line drawn CENTER OF PRESSURE of each joint. The CENTER OF PRESSURE of any joint is the point in which the resultant of the forces acting on that portion of the pier above the joint cuts it. The line of pressure, or of resistance, when drawn in a pier, shows how near the greatest stress on any joint comes to the edges of that joint. It can be drawn by the following method.

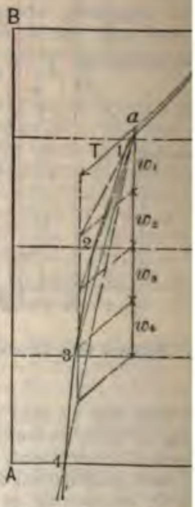


Fig. 6. Line of Press

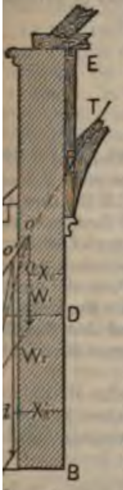
Let $ABCD$ (Fig. 6) be a pier whose LINE OF PRESSURE we wish to draw. Let T be the thrust against the divide the height of the pier into several parts, each 2 or 3 feet hig by the horizontal dotted lines. It is more convenient to make th l in height. Prolong the LINE OF THE THRUST, and draw a vertical h the center of gravity of the pier, intersecting the line of thrust at t. From a lay off to a scale the thrust T, and the weights of the rts of the pier, commencing with the weight of the upper portion. epresents the weight of the portion above the first joint; w_2 repre- weight of the second part; and so on. The sum of the w's will he weight of the whole pier.

---

* This line is called, interchangeably, the line of pressure, the line of resistance-line, etc.

parallelogram, with T and w_1 for its two sides. Draw the diagonal e it beyond the parallelogram, if necessary. Its point of intersection rst joint will be a point in the line of pressure. Draw a second par- with T and $w_1 + w_2$ for its two sides. Draw the diagonal intersecting joint at the point 2. Continue in this way with the rest of the partial weights, the last diagonal intersecting the base AD, in the point 4. Join the points 1, 2, 3 and 4. The resulting broken line $C1234$ is the LINE OF PRESSURE or LINE OF RESISTANCE.

We have taken the simplest case as an example; but the same principles are true for any case. If the line of pressure of the pier at any point falls at a distance from the outside edge of the joint less than ONE-THIRD THE WIDTH OF THE JOINT, the pier is generally considered unsafe.



bility of Wall Buttress

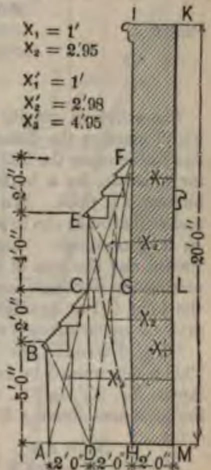


Fig. 8. Stability of Wall and Buttress

The Stability of a Wall and Buttress. By Moments and Graphical Method. The following example illustrates the application of these principles.

Example 2. Let Fig. 7 represent the section of a side wall of a church, with a buttress against it. Opposite the buttress, on the he wall, is a hammer-beam truss, which we will suppose exerts an rust on the wall of the church, amounting to about 9 600 lb. We r consider that the resultant of the thrust acts at P, and at an angle the horizontal. The dimensions of the wall and buttress are given The buttress is 2 ft thick, at right-angles with the plane of the s the buttress the proper SIZE and FORM to enable the wall to resist of the truss?

t point to decide is whether or not the LINE OF PRESSURE cuts the t a safe distance in from C. To ascertain this we must determine n of the center of gravity of the wall and buttress above the joint CD One way to determine this is by the METHOD OF MOMENTS, the F THE AREAS being taken about the line KM *as an axis, or line of* Fig. 8), as already *explained. The distance* X_1 *is, of course, half the f the wall or 1 ft. We next find the center of gravity of the part*

CEFG (Fig. 8) by the method of diagonals; and scaling the distan
it to be 2.95 ft.

The area $CEFG = A_2 = 10$ sq ft; and the area $GIKL = A_1 =$
$A =$ the total area above *CL*.

Then we have

$$\begin{array}{lll} X_1 = 1 \text{ ft} & A_1 = 26 \text{ sq ft} & A_1 \times X_1 = 26 \quad \text{sq ft} \\ X_2 = 2.95 \text{ ft} & A_2 = 10 \text{ sq ft} & A_2 \times X_2 = 29.5 \text{ sq ft} \\ & \overline{A = 36 \text{ sq ft}} & \overline{36)55.5 \text{ sq f}} \\ & & X_0 = 1.5 \text{ ft} \end{array}$$

Expressed in EQUATIONS OF MOMENTS OF AREAS, this may be wri
A representing the total area above the line *CL* (Fig. 8):

$$A \times X_0 = (A_1 \times X_1) + (A_2 \times X_2)$$

Hence,

$$X_0 = \frac{(A_1 \times X_1) + (A_2 \times X_2)}{A}$$

The center of gravity is at a distance 1.5 ft from the line *ED*
Fig. 7 measure the distance $X_0 = 1.5$ ft, and through the point *a*
line intersecting the line of the thrust prolonged at *O*. If the thr
for example, for a buttress 2 ft thick, it will be half that, or
buttress 1 ft thick. We will call the weight of the masonry of whi
and wall is built, 150 lb per cu ft. Then the thrust is equivalent to
cu ft of masonry. Laying this off to a scale from *O*, in the di
thrust, and the area of the masonry, 36 sq ft, from *O* on the vert
pleting the rectangle, and drawing the diagonal, we find that the
the joint *CD* at *t*, within the limits of safety. We must next
LINE OF PRESSURE cuts the base *AB*.

First, determine the position of the center of gravity of the
This is determined by finding, as explained for the distances X_1 ar
tances X_2', X_3', in Fig. 8, and making the following computation
the total area above *AM*.

$$\begin{array}{lll} X_1' = 1 \text{ ft} & A_1' = 40 \text{ sq ft} & A_1 \times X_1 = 40 \quad \text{s} \\ X_2' = 2.98 \text{ ft} & A_2' = 24 \text{ sq ft} & A_2 \times X_2 = 71.52 \text{ s} \\ X_3' = 4.95 \text{ ft} & A_3' = 12 \text{ sq ft} & A_3 \times X_3 = 59.40 \text{ s} \\ & \overline{A' = 76 \text{ sq ft}} & \overline{76)170.92 \text{ s}} \\ & & X_0' = 2.25 \end{array}$$

This, also, may be expressed in EQUATIONS OF MOMENTS OF AREA
for the part above the line *CL*.

Then from the line *EB* (Fig. 7) lay off the distance $X_0' = 2.25$
through *d* a vertical line intersecting the line of the thrust at *O'*.
tical line, measure down from *O'* the whole area 76, to scale, as ex
and from the lower extremity of this line representing the area,
proper angle, the thrust $T = 32$. Draw the line *O'e*, intersecting
This is the point where the LINE OF PRESSURE cuts the base; a
a safe distance in from *A*, the buttress has sufficient stability.
more offsets, we should proceed in the same way, finding wher
PRESSURE cuts the joint at the top of each offset. The reason

LINE OF PRESSURE might cut the base at a safe distance from the while higher up it might come outside of the buttress or too near face, thus making the buttress unstable. The method given in these applicable to piers of any shape or material. If the LINE OF PRES- es an angle of less than 30° with any horizontal joint, the stones joint may SLIDE at this joint, or at least have a strong tendency to DING can be prevented either by doweling, or g the joints. Such conditions, however, are hitectural construction.

bility of a Wall and Buttress. Graphical This same example, which has been solved in ing case partly by MOMENTS and partly by METHODS, can be solved entirely by GRAPHICAL In this case it is not necessary to determine n of the line (the heavy red lines in Figs. 3 and through the center of gravity of the buttress whole. It is necessary, only, to determine nes passing through the cen- ity of the various trapezoids gles into which it has been . To determine the position OF PRESSURE and the various PRESSURE on the different method shown in Fig. 6 may The construction shown in , in which the complete ms of the forces acting at re drawn, may be simplified. e parallelogram, only, or the F THE FORCES acting at each be drawn and the whole n placed at one side of the afterwards transferred to the by means of parallel lines. e joint-planes *FG*, *EJ*, *CK* d calculate the areas of the ts of the wall and buttress, *GF*, *FGJE*, *EJKC*, *CKNB* , Fig. 9. These are respec- ft, 6 sq ft, 16 sq ft, 10 sq ft t. Lay off these areas to a many square units to a linear 1, *w* 1 *w* 2, *w* 2 *w* 3, *w* 3 *w* 4 and *w* 4 *w* 5, along the line *KM*, begin- point of application of the THRUST. Lay off, at the same scale, the or one foot of thickness of the wall, and let this thrust be 4 800 lb. , *Ow* 2, *Ow* 3, etc. Then *Ow* 1 will be the resultant of the thrust ght of the buttress above the joint *FG*, *Ow* 2 will be the resultant of ultant and the weight of that part of the buttress between the joints , and so on until *Ow* 5 is reached, which is the resultant of the total he buttress and the thrust as well as the resultant of the rectangle the previous resultant. Prolong the thrust *OP*, *until it cuts the* ine through the center *of gravity of the first rectangle IKGF, at a.* *is point draw a (green) line parallel to Ow 1 and prolong it backward*

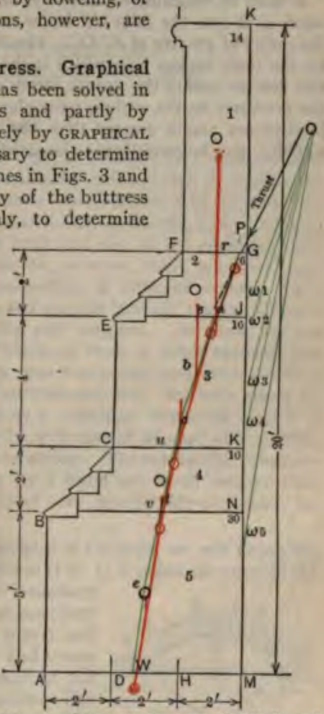


Fig. 9. Line of Pressure in Wall and Buttress

the center of gravity of *EJKC*. Finally, join the various CENTERS OF
by the (red) broken line, which is the LINE OF PRESSURE in the butt
this line lies within the MIDDLE THIRD of the construction, and the res
the pressures on the various joint-planes do not make with the norma
joint-planes angles greater than the ANGLE OF FRICTION, the condi
stability may be considered to be satisfied.

CHAPTER VIII

THE STABILITY OF MASONRY ARCHES

BY
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1. Arches

intel and the Arch. When an opening is made in a masonry wall ssary to provide some means of spanning such opening to support the osed masonry. Two methods have been employed by constructors irpose. The first involves the use of the BEAM, GIRDER, CAP, or LINTEL, econd the throwing of an ARCH from one side of the opening to the INTELS are made of various materials, as wood, stone, reinforced con- t iron and steel, and have cross-sections of different shapes. They l across the tops of the openings and transfer laterally the loads above, ERTICAL REACTIONS, only, in the side supports. An ARCH, on the is a particular arrangement of blocks of stone or other material, put generally along a curved line, in such a way that they resist the load ncing of certain THRUSTS and COUNTERTHRUSTS. An arch exerts on rts an OUTWARD THRUST as well as a VERTICAL PRESSURE; and it is ard thrust which requires that the arch should be used with caution abutments are not amply large and strong. The mechanical principles in the spanning of an opening by a lintel are much simpler than the arch and, historically, the lintel very considerably antedates the

tions. Before taking up the principles of the arch, we will define the terms relating to it. The distance *ec* (Fig. 1) is called the SPAN of the the rise; *b*, the CROWN; the lower boundary he SOFFIT or INTRADOS; the outer boundary BACK or EXTRADOS. The terms SOFFIT and also applied to the entire lower and upper faces of the whole arch. The sides of the ch are seen are called the FACES. The which the arch itself is composed are called s; the center one, *K*, is called the KEYSTONE; lowest ones, *SS*, the SPRINGERS. In SEG- rches, or those of which the intrados is not te semicircle, the springers generally rest upon two stones, as *RR*, which r upper surfaces cut to receive them; these stones are called SKEWBACKS. connecting the lower edges of the springers is called the SPRINGING- sides of the arch are called the HAUNCHES; and the loads in the tri- paces, between the haunches and a horizontal line drawn from the e called the SPANDRELS. The blocks of masonry, or other material, port two successive arches, are called PIERS; and the extreme blocks, the case of stone bridges, generally support, *on one side, embank-* earth, are called *ABUTMENTS. A pier strong enough to resist the one of two successive arches, in case the other one falls down,* is some-

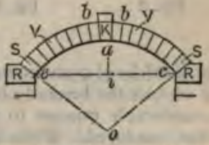

Fig. 1. Diagram of Segmental Arch

an arch of a certain form and of certain dimensions will be stable
under its load. The SEMICIRCULAR and SEGMENTAL forms of arches
best as regards stability, and are the simplest to construct. ELLIPT
THREE-CENTERED arches are not as strong as circular arches, and sh
be used where they can be given all the strength desirable.

The Strength of an Arch depends very much upon the care with w
built and upon the quality of the materials. In stone arches, special ca
be taken to cut and lay the beds of stones accurately, and to make
joints thin and close, in order that the arches may be stressed as little a
in settling. To insure this, arches are sometimes built DRY, grout
mortar being afterwards run into the joints; but the advantage of thi
is doubtful.

Brick Arches.* (See Figs. 2, 3, 4 and 5.) These may be built
WEDGE-SHAPED bricks, molded or rubbed so as to fit to the radius of t

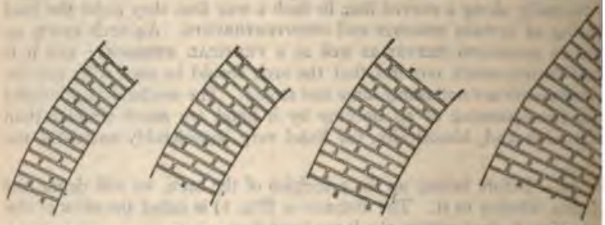

Fig. 2 Fig. 3 Fig. 4 Fig. 5

Brick Arches

or of bricks of COMMON SHAPE. The former method is undoubtedly th
it enables the bricks to be thoroughly bonded, as in a wall; but, as it
considerable expense to make the bricks of the proper shape, it is
dom employed. When bricks of the ordinary shape are used, they
commodated to the curved figure of the arch by making the bed-joint

This may be done either by thickening with pieces of slate the ıe outer ring of a pair of half-brick rings, so that there will be the ber of courses of stretchers in each ring between two courses of r by placing the courses of headers at such distances apart, that ıch pair of them there will be one course of stretchers more in the in the inner ring. The former method is best suited to arches of s; the latter, to those of short radius. HOOP-IRON laid around the een half-brick rings, as well as longitudinally and radially, is very strengthening brick arches. The bands of hoop-iron which traverse ıdially may also be bent, and prolonged into the bed-joints of the ıd spandrels. By the aid of HOOP-IRON BOND, Sir Marc-Isambard lt a half-arch of bricks, laid in strong cement mortar, which stood, from its abutment like a bracket to the distance of 60 ft, until it was by the undermining of its foundations.

y requirements in the New York City Building Laws for brick and s is that "openings for doors and windows in all buildings shall have sufficient arches of stone, brick, or terra-cotta, well built and keyed ood and sufficient abutments."

r the Radius of Brick Arches. A good RULE for the radius of brick arches over windows, doors and other small openings is to RADIUS EQUAL TO THE THE OPENING. This od rise to the arch and proportion. In com- twork, when no par- rchitectural effect is ıch as in the rowlock wn over the openings walls, a RULE in very se is to make the RISE at the crown AN INCH FOR EVERY FOOT OF

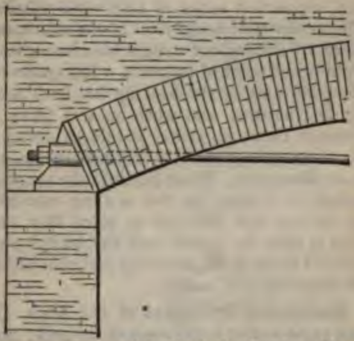

Fig. 6. Segmental Brick Arch, Cast-iron Skew-back and Wrought-iron Tie-rod

tal Arches with Tie- is often desirable to ngs in a wall by means when there are not a number of abutments and the thrusts. In is kind each arch can be sprung from two cast-iron SKEWBACKS, held IRON RODS, as is shown in Fig. 6. When this is done, it is necessary ion the size of the rods to the THRUST of the arch. The HORIZONTAL the arch may be very closely determined by the following formula:

$$\text{Horizontal thrust} = \frac{\text{load on arch} \times \text{span}}{8 \times \text{rise of arch in feet}}$$

ad is concentrated at the center of the arch, the thrust will be twice by this formula.

SIONAL STRESS in the rod or rods will equal the HORIZONTAL THRUST h and if there are two rods, the stress in each will be *one-half the* there are three rods, then *each must resist one-third the thrust.* the stresses *in the rods, the size of each may be determined from* *Chapter XI.*

wall; and since ordinary brickwork weighs about 115 lb per cu ft, will be about 10 867 lb. Substituting these values in the formula,

$$\text{The horizontal thrust} = \frac{10\,867 \times 15}{8 \times 1.5} = 13\,584 \text{ lb}$$

Looking in Table II, page 388, it appears that one 1½-in or two 1 round, wrought-iron rods, or one 1⅛-in or two ¾-in round, upset, should be used.

Centers for Arches. A CENTER is a temporary structure, ge timber, on which the voussoirs of an arch are supported while the ar built. It consists of parallel frames or ribs, placed at convenient apart, curved on the outside to a line parallel to that of the soffit o and supporting series of transverse planks, upon which the arch-st The center commonly used is one which can be lowered, or STRUCK piece, by driving out wedges from below it, so as to remove at once t from every point of the arch. The center of an arch should not be st the solid part of the backing has been built and the mortar has h set and harden; and when an arch forms one of a series of arches between them, no center should be struck so as to leave a pier wit abutting against one side of it only, unless the pier has sufficient stabi as an abutment. When possible, the STRIKING of the center of large br should be delayed for two or three months after the arch is built, a the period that they are in place they should be EASED from tim This is done by EASING OUT the wedges under the centers a little at as to let them down gradually and thus adjust any slight settling or sh the masonry as it occurs.

Mechanical Principles of the Arch. In designing an arch, the tion to be settled is the FORM of the arch; and in regard to this, as alre there is generally little choice. When the abutments are of ample siz MENTAL ARCH is the strongest; but when it is necessary to make the of the arch as small as possible, the SEMICIRCULAR or the POINTED AR be used.

applies to both CIRCULAR and ELLIPTICAL ARCHES and is as follows. n proportional between the inside radius at the crown, and 0.12 of single arch, and 0.17 of a foot for an arch forming one of a series:

feet of keystone for single arch $= \sqrt{(0.12 \times \text{radius at crown})}$
feet of keystone for arch of a series $= \sqrt{(0.17 \times \text{radius at crown})}$

ensions given by this formula seem to agree very well with those sed in practice in arches of a certain kind. The formula, however, ame depth of keystone for spans of any length, provided the radius ; and in this particular it would seem that the rule is not satisfactory.

ine's Formula for Depth of Keystone. Trautwine, from calcu- de for a large number of arches, deduced a formula for the depth of which seems to agree with theory more closely than Rankine's formula. for CUT STONE,

$$\text{Depth of key in feet} = \left(\frac{\sqrt{\text{radius} + \text{half span}}}{4}\right) + 0.2 \text{ ft}$$

OND-CLASS work this depth may be increased about one-eighth part, KWORK OR FAIR RUBBLE, about one-third.

for Depths of Keystones. Table I gives a few examples of the THE KEYSTONES of some bridges, together with the depths which required by Trautwine's or Rankine's formula. From this table it at the results of both formulas agree very well with dimensions used ractice.

le I. Depths of Keystones of Some Arches of Circular Arc

r location of ucture	Span	Rise	Radius	Actual depth of key	Calculated depth of key		Engineer
					Trautwine's Rule	Rankine's Rule	
	ft	ft	ft	ft	ft	ft	
hn, Washing- aeduct.......	220.0	57.25	134.25	4.16	4.11	4.00	Meigs
nor bridge , England....	200.0	42.00	140.00	4.00	4.07	4.10	Harfley
aria, Turin, ...	148.0	18.00	160.10	4.92	4.03	4.38	Mosca
nd, England	118.0	38.00	64.80	3.50	3.00	2.79	Telford
dge, Scotland, ies...........	90.0	30.00	48.90	3.00	2.62	2.88	Telford
idge, Phila- & Reading d............	78.0	25.00	43.00	3.00	2.46	2.27	Steele
St. bridge, lphia, brick nt............	60.0	18.00	34.00	2.50	2.20	2.00*	Kneass
hia & Read- lroad........	44.0	8.00	34.30	2.50	2.08	2.02	Steele
hia & Read- lroad........	31.2	5.00	*26.80*	*1.66*	*1.83*	*1.79*	*Steele*

* *For first-class cut-stone work.*

Table II* gives the DEPTHS OF KEYSTONES for arches of first-class cu according to Trautwine's Formula. For second-class cut stone, add abo eighth part and for fair rubble or for brickwork about one-third part, a with formula.

Table II. Depths of Keystones for Arches of First-Class Cut-Ston Masonry

Span	Rise, in parts of the span					
	½	⅓	¼	⅕	⅙	⅛
ft	ft	ft	ft	ft	ft	ft
2	0.55	0.56	0.58	0.60	0.61	0.64
4	0.70	0.72	0.74	0.76	0.79	0.83
6	0.81	0.83	0.86	0.89	0.92	0.97
8	0.91	0.93	0.96	1.00	1.03	1.09
10	0.99	1.01	1.04	1.07	1.11	1.18
15	1.17	1.19	1.22	1.26	1.30	1.40
20	1.32	1.35	1.38	1.43	1.48	1.59
25	1.45	1.48	1.53	1.58	1.64	1.76
30	1.57	1.60	1.65	1.71	1.78	1.91
35	1.68	1.70	1.76	1.83	1.90	2.04
40	1.78	1.81	1.88	1.95	2.03	2.18
50	1.97	2.00	2.08	2.16	2.25	2.41
60	2.14	2.18	2.26	2.35	2.44	2.62
80	2.44	2.49	2.58	2.68	2.78	2.98
100	2.70	2.75	2.86	2.97	3.09	3.32
120	2.94	2.99	3.10	3.22	3.35	3.61
140	3.16	3.21	3.33	3.46	3.60	3.87
160	3.36	3.44	3.58	3.72	3.87	4.17
180	3.56	3.63	3.75	3.90	4.06	4.38
200	3.74	3.81	3.95	4.12	4.29	....
220	3.91	4.00	4.13	4.30	4.48	....
240	4.07	4.15	4.30	4.48	....	....
260	4.23	4.31	4.47	4.66	....	....
280	4.38	4.46	4.63	....	....	....
300	4.53	4.62	4.80	....	....	....

Example 2. Having decided what the thickness of the arch-ring w remains to determine whether such an arch would be stable if buil following example will illustrate the method of determining this.

Consider an unloaded semicircular arch of 20-ft span.

First, to find the depth of the keystone, we will use Rankine's Formu

$$\text{Depth of key} = \sqrt{0.12 \times 10} = \sqrt{1.2} = 1.1 \text{ ft}$$

Trautwine's Formula gives nearly the same result,

$$\text{Depth of key} = \frac{\sqrt{10 + 10}}{4} + 0.2 \text{ ft} = 1.3 \text{ ft}$$

But if we should compute the stability of a 20-ft semicircular arch keystone 1.3 ft deep, we should find that the arch is very unstable; h this case, we cannot use the formula and must act upon our own ju In the opinion of the author, the arch-ring of such an arch should be 2½ ft deep and the stability of the arch should be tested for that th In all calculations on the arch, it is customary to consider it 1 ft t its face. This allows the AREAS OF THE FACES to be substituted L WEIGHTS of the voussoirs and their loads. This method was iscussion of Retaining-Walls, Chapter IV, and Piers and But- er VII. Furthermore, it is evident that if an arch 1 ft thick is mber of arches of the same dimensions built alongside of it would determining the stability of masonry arches it is also customary increase in the strength of the arch from the mortar in the joints, rds, to consider the arch as laid up dry.

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* Taken from The Civil Engineer's Pocket-Book, John C. Trautwin

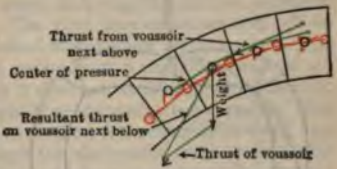
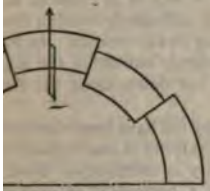

Thrust from voussoir next above
Center of pressure
Weight
Resultant thrust on voussoir next below
←Thrust of voussoir

Fig. 7. Equilibrium of Forces on Voussoir

etermination of the Stability of Arches. An arch has already as a particular arrangement of blocks of stone or other material, being called the vous- the sake of simplicity UNLOADED ARCH. In each voussoir is sub- action of three forces, t that it receives from next above it in the the force of gravitation, ight and (3) the reaction nt thrust. The first two ne into one and form at this voussoir exerts on the one next below it in the arch-ring e points in which these various thrusts cut the joints are called the RESSURE of the joints, while the line joining these centers of pres- the LINE OF PRESSURE or LINE OF RESISTANCE.* In order that an bsolutely stable, this line of resistance must fall within the MIDDLE arch-ring. (See Theorem of the Middle Third, Chapter IV.) If able the centers of pressure on the various joint-lines are within ird of the voussoir-depths and the angles made by the different the normals to the joints are less than the ANGLE OF FRICTION of of which the arch is constructed. If these conditions are not ful- ITERIA OF SAFETY, explained in Chapter VII in the discussion of of a Buttress, will not be satisfied; and at any joint where these not obtain, the voussoir above the joint will tend to SLIDE along ne if the angle made by the thrust with a normal to the joint is the angle of friction. If the center of pressure lies outside the there will be a tendency for the voussoir to OVERTURN. When ies reach extreme limits actual FAILURE may occur. Figures 8, 9, ustrate some of the ways in which an arch *may fail, Figs. 8 and 9,*



ure of Semicircular Arch. ches Sliding Down

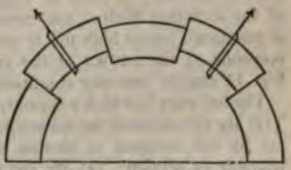


Fig. 9. Failure of Semicircular Arch. Haunches Sliding Up

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is called, *interchangeably, the LINE OF PRESSURE, the LINE OF RESISTANCE, -LINE, etc.*

the joints an angle greater than the angle of friction, the voussoirs at t

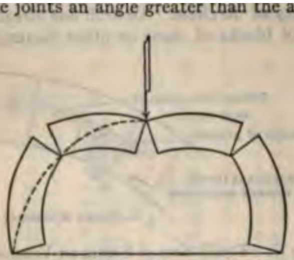


Fig. 10. Failure of Semicircular Arch. Opening of Arch-ring

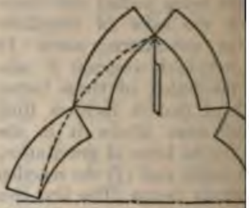


Fig. 11. Failure of Pointed Opening of Arch-ring

would tend to slide inward on their joint-planes, forcing outward the at the spring and crown of the arch. Fig. 9 shows how failure of the ar occur under similar conditions, but with the line of resistance turning upward instead of downward. In these two cases it is conceivable though the RESISTANT THRUST at the joint where failure takes place m angle with the normal greater than the angle of friction, its point of ap is still within the middle third of the joint.

Figs. 10 and 11, on the contrary, illustrate methods of failure in w though the angle made by the thrust may be such as to cause no SLIPPIN joint on another, its point of application is sufficiently outside the mid of the arch-ring itself at the crown to cause OVERTURNING. In Fig. 10 of resistance passes high up, or perhaps entirely outside of the arch-rin voussoirs at the CROWN of the arch and low down along the HAUNCI Fig. 11 exactly contrary conditions exist.

The ten ways in which a masonry arch may fail have been classified as f "(1) By CRUSHING of the masonry; (2) By SLIDING of one voussoir upon en arch is stable, it is necessary to find the TRUE LINE OF RESISTANCE ing to the conditions of loading, form and dimensions of that par- h. It is always possible, in every arch-ring, to pass one MAXIMUM NIMUM LINE OF RESISTANCE. The TRUE LINE OF RESISTANCE will lie between these two. The method of procedure, therefore, is to pass , a line of resistance, either a maximum or a minimum one, and see s within the middle third. If it does not, as it may not be the true tance, it does not mean necessarily that the arch is not stable. The hen, is to note where it departs farthest from the middle third, and econd line of resistance through the same point on the crown-joint int on the line of the middle third where the original line departs om the middle third. If this second line of resistance remains within third it is reasonable to assume that the arch is stable. In these erations it is only necessary to consider half the arch when the loading rical, and this is usually the case in architectural problems. The F VOUSSOIRS, also, into which we divide the half-arch, is immaterial nts need not coincide with those of the actual arch.

to pass a line of resistance through an arch-ring, the THRUST exerted er half AT THE CROWN-JOINT on the half-arch is first determined. is then combined with the resultant of the weight of the first voussoir d to determine the thrust exerted by this voussoir on the one next d this thrust, in turn, is combined in the same way with the resultant ht and the load of the second voussoir, and so on down to the spring- or each succeeding voussoir. The points in which the various lines g the thrusts cut the joints are known as the CENTERS OF PRESSURE, e joining them is the LINE OF PRESSURE or LINE OF RESISTANCE. In this operation, the CENTER OF GRAVITY of each voussoir as well as ssing through the center of gravity of the whole half-arch must be The face of each voussoir may be considered a TRAPEZOID, and any methods for finding the center of gravity of this figure may be used the center of gravity of each voussoir. The method of dividing the nto TRIANGLES is here employed and is shown at the side of the arch (See, also, Chapter VI, and VII, page 298.) As the determination tion of the line passing through the center of gravity of the half-arch lem of finding the RESULTANT OF A SYSTEM OF PARALLEL FORCES, the volving the drawing of the EQUILIBRIUM-POLYGON may be used. convenient way to determine the stability of an arch is to use the ETHOD. The STEPS in this method are outlined in the preceding para- ach of the operations will now be considered in detail.

p. Draw one-half the arch to as large a scale as convenient, and nto voussoirs of equal size. In the example shown in Fig. 12, the divided into ten voussoirs of equal face-areas. As already *pointed* ot necessary that these should *represent the actual voussoirs of which* *built. Next, the face-area of each of these voussoirs* is to be found.

Where the arch-ring is divided into voussoirs of equal size, this is
done by computing the total area of the arch-ring and dividing this
by the number of voussoirs. The FORMULA for finding the area of
arch-ring is as follows:

$$\text{Area in square feet} = 0.7854\,(r^2 - r_1^2)$$

In this formula r is the outside radius and r_1 the inside radius in fe
In this problem, for example, if the

$$\text{Area of the arch-ring} = 0.7854\,(12.5^2 - 10^2) = 44.2 \text{ sq ft}$$

as there are ten equal voussoirs, the area of each voussoir is 4.42
ing drawn out one-half of the arch-ring, divide the crown-joint into
parts, and
of $O'E$ and
the arcs d
arch-ring in

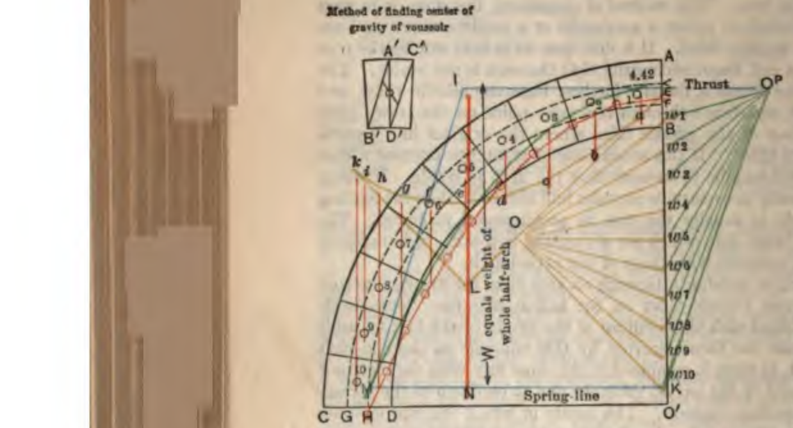


Fig. 12. Line of Pressure in Unloaded Semicircular Arch-ring

Second St
the points
through wh
a MINIMUM
SISTANCE.
F and G, th
a MAXIMUM
SISTANCE ca
could equall
been chosen
be noted t
loaded sem
arch is more
by opening
trados at
and at the
the spring,
fore, in thi
line of resist
ably passes nearer the OUTER THIRD at the CROWN and nearer the I
at the SPRING. To determine this MINIMUM LINE OF RESISTANCE t
THRUST, applied at the point E of the crown-joint, must first be det

The half-arch is in equilibrium under the action of three force
THRUST AT THE CROWN, acting horizontally, applied at the point E and
the half-arch from overturning inward; (2) the WEIGHT OF THE
considered as a vertical force, acting through its center of gravity a
to overturn it inwards about the point D; and (3) A FORCE EQUAL
SITE TO THE RESULTANT of these two forces and passing from H to
intersection of the weight-line through the center of gravity of the
with the line of action of the thrust at the crown, prolonged. It is th
to construct the TRIANGLE OF THESE THREE FORCES and determine the
of the thrusts, when the position of the weight-line of the half-arc
mined. It is first necessary to draw a vertical line through the cente
of each voussoir. The center of gravity of one of the voussoirs ma
by the METHOD OF TRIANGLES, as shown in the supplementary figure
of the arch-ring.

Having determined the positions of the centers of gravity of the

voussoirs as shown. From the point E (Fig. 12) lay off verti-
so many SQUARE UNITS TO A LINEAR UNIT, the area of each
w the other, commencing with the top voussoir. The length
ll then equal the total area of the arch-ring. From E and
45° lines intersecting at O. Draw Ow 1, Ow 2, Ow 3, etc.
ntersects the first vertical line through the center of gravity
ir at a, draw a line parallel to Ow 1, intersecting the second
aw bc parallel to Ow 2, cd parallel to Ow 3 and so on to k.
to Ow 10 and prolong it downward until it intersects EO pro-
vertical line drawn through L will pass through the center of
f arch-ring. This is an application to a practical problem of
ding, by the EQUILIBRIUM-POLYGON, the line of action of the
STEM OF PARALLEL FORCES. The weights of the individual
g parallel vertical lines and the weight of the half-arch is their
itude.

o determine the THRUST AT THE CROWN and the REACTION
aw a horizontal line through E, the upper part of the middle
cal line through L, the two lines intersecting at I (Fig. 12).
e stable, it is, in general, considered necessary for the LINE
pass within the MIDDLE THIRD. First, assume that the line
istance starts at E and comes out at H. Draw a line IH
he line of action of the resultant of the thrust at the crown
f the half-arch, and draw, also, a horizontal line opposite the
en N and M. This horizontal line MN represents the magni-
ontal thrust at the crown, for INM is the TRIANGLE OF THE
equilibrium, the THRUST at the crown, the WEIGHT of the
REACTION at the spring. Draw w 10 O^P parallel to HI, and
)^Pw 2, O^Pw 3, etc. O^PE, equal to NM, is the thrust at the
)^P, equal to MI, the reaction at the spring. INM and EKO^P
es.

It is required next, to determine the LINE OF RESISTANCE
-ring. The thrust at E is combined with the weight of the
ir resultant is found and in turn combined with the weight of
ir; and so on for all the voussoirs. The intersections of these
e joint-lines are the CENTERS OF PRESSURE; the line joining
ressure is the LINE OF RESISTANCE.
ts could be determined by drawing a series of PARALLELO-
s over each voussoir. This would complicate the figure and
ry labor. It is found more convenient to draw the TRIANGLES
fter the other, at the right-hand side of the figure and then
lts thus obtained by means of parallel lines to the figure
s the weights of the voussoirs have already been laid off along
w 1, w 2, w 3, w 4, w 5, etc.
e point where O^PE prolonged intersects the first vertical in
1, draw a (green) line to the second vertical, parallel to O^Pw_1;
a (green) line to the third vertical, parallel to O^Pw 2 and so
e should pass through H. Join the various points, where these
the joints at the centers of pressure, by the broken (red) line.
wn is the LINE OF RESISTANCE. If this line lies entirely within
o of the arch-ring, the arch may be considered to be stable.
the line of resistance passes not only outside *of the middle*
tside of the arch-ring *itself; it is still possible that the arch*
This is the case in Fig. 12 and we will next determine if a

line of resistance can be drawn which will remain within the limits of third of the arch-ring.

Fifth Step. The Second Trial. Reproducing the condition of Fig. 13, without the construction lines, it is seen that the LINE OF leaves the arch-ring enters it again at S furthest from it at U a perpendicular is straight line joinin points *R* and *S*, dicular line *VW*, call OF FRACTURE, will mately the trace along which, with resistance under the arch will tend sumedly by TURNI the right about th This shows that the THE CROWN, assu applied at the point sufficient intensity equilibrium about sufficient intensity equilibrium about SECOND THRUST, intensity to maintain equilibrium about *V*, or better, about *X*, ca at *E* without being so great in magnitude that it will OVERTURN OUTWARD about *G*, or some other point on the outer line of the mi

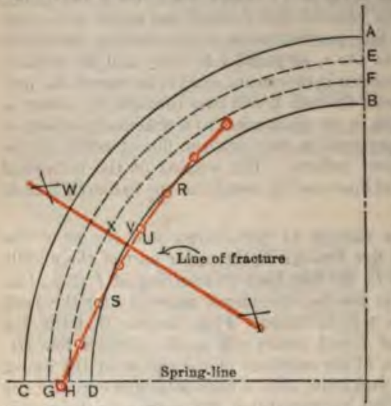


Fig. 13. Line of Fracture in Unloaded Semicircular Arch-ring

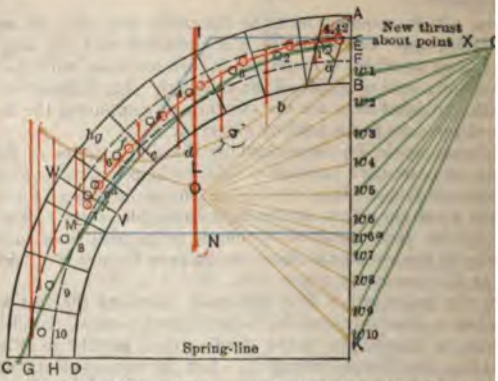


Fig. 14. Second Line of Pressure in Unloaded Semicircular Arch

is reasonable to conclude that the line of resistance resulting from very nearly the TRUE LINE OF RESISTANCE in the arch-ring and t is stable.

In order to determine this NEW LINE OF RESISTANCE the NEW THR nust be found (Fig. 14). The preliminary steps required for this are as before until the seventh voussoir is reached. This is divided into ssoirs by the line VW (Fig. 14), one being $w6\ w6^a$ and the other the r of this seventh voussoir, and this division must be allowed for along line EK, at $w6\ w6^a$. The line $w6\ w6^a$ represents the area of vous- nd the line $w6^a\ w7$ the area of the remainder of the seventh voussoir. rtical line IL, passing through the center of gravity of that part of the above the line VW, is found by prolonging backwards the line *hg*, o $O\,w6^a$, until it intersects OE at L. To find the NEW THRUST AT THE y completing the TRIANGLE OF FORCES for this thrust and the force d opposite to their resultant, the inclined (blue) line must be drawn the point X and the horizontal (blue) line through $w6^a$. The new thrust s before NM, equal to O^pE. This thrust is laid off at O^pE, the (green) w 1, O^pw 2, O^pw 3, etc., being drawn as before and the new line of re- being drawn through the points where the parallels to these (green)

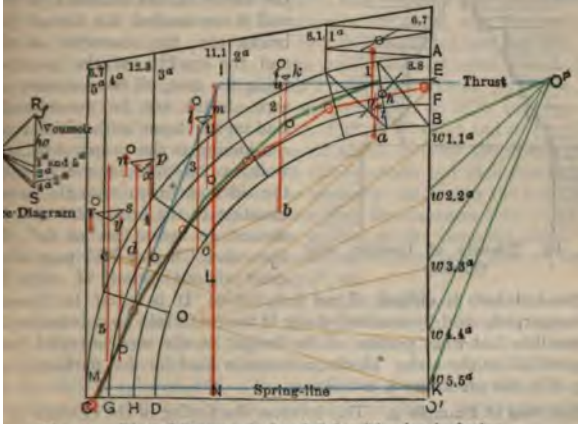


Fig. 15. Line of Pressure in Loaded Semicircular Arch-ring

the joints. This NEW LINE OF RESISTANCE, if drawn correctly, should ough X. It lies within the middle third, except for a short distance at ging, and hence it is justifiable to consider the arch stable. If it had utside the middle third to any great extent, in this second trial, this tion would not have been justified.

discussion explains the method of determining the stability of an UN- SEMICIRCULAR ARCH. Such cases very seldom occur in practice, but ve to illustrate the methods which apply generally to all other cases. ADED ARCH-RINGS there is slight difference in the method of determining tion of the center of gravity.

ple 3. A LOADED OR SURCHARGED SEMICIRCULAR ARCH (Fig. 15) will be ed next. Assume the same arch shown in Figs. 12, 13 and 14, and sup- o be loaded with a wall of masonry of the same thickness and weight re foot as that of the arch-ring, the upper *surface of the wall being an* plane, 1 ft above *the arch-ring at the crown, and 8 ft above it at the* *The assumption of the particular load in this case is a purely arbitrary* total superimposed masonry bears upon the arch-ring.

It is a matter of common observation that if an opening is made especially in a wall that has stood for some time, the major portion of th above this opening is self-supporting, limited portions only, bounded what irregular line, falling down into the opening, as shown in Fig profile of this boundary-line dep the nature of the material of wall is constructed, the size of bricks, etc., the character of and the quality of the mort being the case, all the masonry arch should not be considere load on it. Some authorities re considering as the proper load, work, a TRIANGULAR PART of the sides of which triangle ha clination to the horizontal of 4 assume an inclination of 60° (See, also, Chapter XV, page 6 exact determination of this mechanical laws is difficult if not impossible. It is better to cons case separately and by a careful study of the conditions to determine as possible just what portion of the weight of the superimposed m transmitted to the arch. Having assumed a load for this particular (Fig. 15), the procedure is as follows:

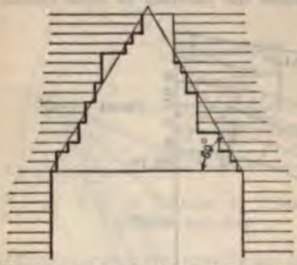


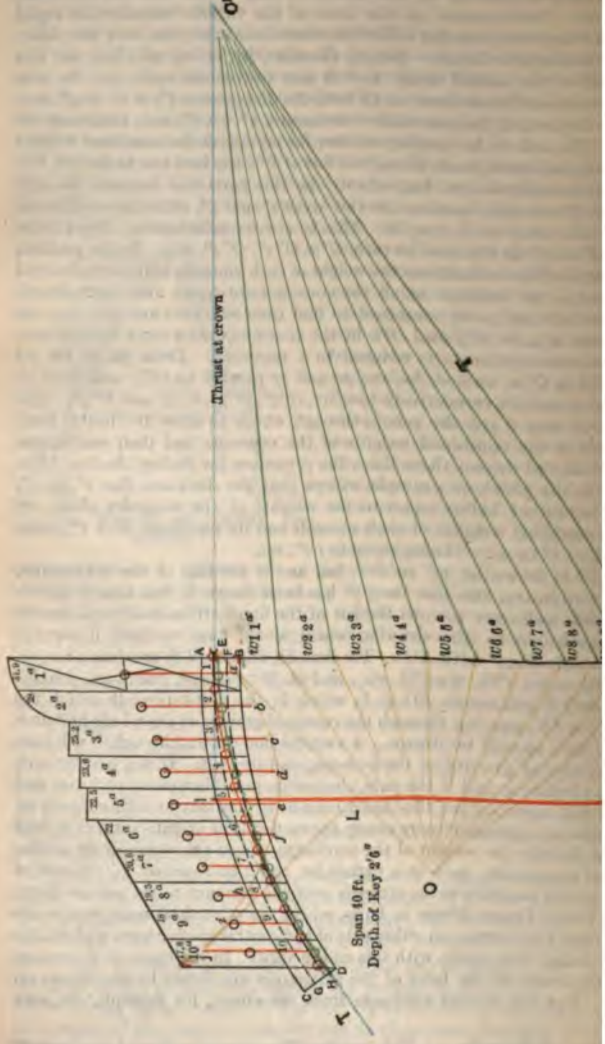
Fig. 16. Triangle of Loading over Opening

First Step of Example 3. This involves the finding of the CENTER O of the ARCH-RING AND LOAD COMBINED. Divide the arch-ring into five of equal size. In this case the area of each voussoir is equal to 44.2 sq 8.8 sq ft. (See under First Step, Fig. 12, preceding example.) The or load, also, is divided into five parts, not necessarily equal, by dra

Fig. 15. The method used, that of the EQUILIBRIUM-POLYGON, e as that employed in the previous example to find the line passing e center of gravity of the half-arch, only in this case the forces are two. Furthermore, as the areas of the various voussoirs are equal le to superimpose the different FORCE-DIAGRAMS, one over the other, e considerable labor. Begin, therefore, by laying off along the line left of the loaded arch, and at any convenient scale, *fw*, the area f a voussoir; then from *w*, in turn, the distances $w\ 1^a$, $w\ 2^a$, $w\ 3^a$, etc., g the areas of the successive surcharges, 1^a, 2^a, 3^a, etc., always at the . The scale to be employed later for laying off the combined weights ssoirs and their loads along the line *AK* is the best one to choose, but nce in scales is not important. In this particular instance the two nd 5^a coincide because the two areas 1^a and 5^a, although of different e each equal to 6.7 sq ft. This is a mere coincidence. Next draw O'' at 45° to *RS*, and in turn, $O''w$, $O''1^a$, $O''2^a$, etc. As the problem ents itself is to combine the weight of each voussoir with its individual and as the weights of all the voussoirs are equal, and, furthermore, es which are to be combined to find their resultant are only two, the LINES or RAYS $O''f$ and $O''w$ in the FORCE-DIAGRAM serve in each case, UNICULAR POLYGON is reduced to a TRIANGLE. Draw *gh*, *ik*, *lm*, *np* allel to $O''w$, and *ht*, *ku*, *mv*, *px* and *sy* parallel to $O''f$; and draw *gt*, nd *ry* parallel respectively to $O''1^a$, $O''2^a$, $O''3^a$, $O''4^a$ and $O''5^a$. The , *v*, *x* and *y* are the points through which to draw the heavy (red) tion of the combined weights of the voussoirs and their surcharges. found and drawn these lines, the procedure for finding the line *IN* is s in the previous example, except that the distances $Ew1\ 1^a$, $w1\ 1^a$, ., instead of being equal to the weights of the voussoirs alone, are e combined weights of each voussoir and its surcharge, $Ew1\ 1^a$, being 1^a, $w1\ 1^a$ to $w2\ 2^a$ being equal to $f\ 2^a$, etc.

e *EO* is drawn at 45° to *AO′*, but as the position of the POLE-POINT, ely arbitrary, the line $Ow\ 5\ 5^a$ has been drawn in this case in such a *O* falls well over toward the left of the figure, thus avoiding a certain f confusion in the drawing which would have resulted if $Ow\ 5\ 5^a$ an angle of 45° with *AO′*. The lines *ab*, *bc*, *cd* and *de* are drawn respecallel to $w1\ 1^aO$, $w2\ 2^aO$, etc., and *eL* is produced backward parallel until it intersects *EO* at *L*, which is the point through which the) line *IN*, passing through the center of gravity of the whole half-arch rcharge, should be drawn. A vertical line drawn through *L* will pass e center of gravity of the arch-ring and its load. If this were an arch or a building and if the only abutments possible were of such size and it was essential for the thrust exerted by the last or fifth voussoir on ments to approach more nearly the vertical, the architectural expedient g slightly the weight of the surcharge, 5^a, on this voussoir by adding e of ornament, such as a cartouche, could be resorted to. A case of n actual practice is the archway over the entrance to the service-courte Grand Opera House in Paris, where the pyramidal stone ornaments nount the cornice on either side of the central motive were added after l design was made, with this end in view. In the example illustrated the areas of the faces of the surcharges are shown by the figures on s. For the second surcharge from the crown, for example, the area t.

Step of Example 3. *This involves the determination of the* THRUST *own and the* LINE OF RESISTANCE. *The method of finding this thrust*


O^P
Thrust at crown
w1 1^a
w2 2^a
w3 3^a
w4 4^a
w5 5^a
w6 6^a
w7 7^a
w8 8^a
A
E
F
B
C
G
H
D
J
L
O
T
1^a
2^a
3^a
4^a
5^a
6^a
7^a
8^a
9^a
10^a
Span 40 ft.
Depth of Key 2'6"

is example may be used, also, for a SEMIELLIPTICAL ARCH.

4. This example (Fig. 17) illustrates the application of the preced- ds, with some variations, to the determination of the position of the gravity of a LOADED SEGMENTAL ARCH, the thrusts at the crown and the line of pressure or resistance through the arch-ring. In this case, dividing the arch-ring into a certain number of voussoirs with joints from a center and considering the surcharge on each individual voussoir, d of dividing the arch-ring and its load into VERTICAL SLICES, in this et wide, and computing the areas of the entire slices has been adopted. mputed the areas of the slices, including in each case the combined e sliced part of the arch-ring and its surcharge, we lay them off in order o a convenient scale, and then proceed as in the previous examples. ining steps required to determine the thrusts at the crown and at the the line of resistance are also the same as explained in the foregoing s. In a FLAT SEGMENTAL ARCH there is practically no need of dividing ring into voussoirs by JOINTS RADIATING FROM A CENTER, in order to its stability. Of course, when built, they must be made to radiate. shows the GRAPHICAL ANALYSIS of an arch of 40-ft span and carrying ½ ft high at the crown. The depth of the arch-ring is 2 ft 6 in. It is the line of resistance lies entirely within the middle third, and that s therefore stable. It is to be noted that the line of resistance in a L ARCH should be drawn through the LOWER or INNER EDGE of the ird at the springing. It is to be noted, also, that the horizontal thrust wn and the thrust *T* against the supports are very great when com- those in a SEMICIRCULAR ARCH; and hence, although the SEGMENTAL e stronger of the two, it requires much heavier abutments. The fore- nples serve to show the various methods of determining the stability s of any arch used in buildings.

ed-Concrete Arches. Many arches are now built of concrete rein- steel ribs. Such arches, however, are much more common in civil than in architectural structures and so hardly come within the f an architect's handbook. Many comprehensive and practical

CHAPTER IX

REACTIONS AND BENDING MOMENTS FOR B

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1. Reactions for Beams

Definition of Reaction. One of the fundamental principles of sta
rium is that the sum of all the forces acting upon a body in one directi
balanced by the sum of another set of forces acting in the opposit
Therefore, in the case of a beam or girder, the loads acting downwa
balanced by an equal set of forces at the supports, acting upward.
ward forces are called THRUSTS, or REACTIONS and in computing th
of beams one of the first steps is to determine them, since the loads
given in intensity and position.

The Principle of Moments. The reactions may be determin
application of another fundamental principle of static equilibrium
acting in the same plane. The algebraic sum of the moments of all

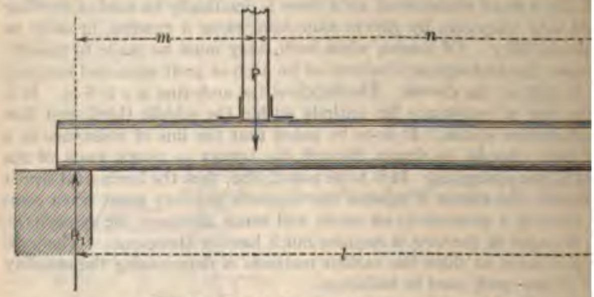

Fig. 1. Simple Beam. One Concentrated Load

taken about any point in the plane in which they act must be zero. T
OF A FORCE about a point is the product of the magnitude or intensity
by the perpendicular distance between the LINE OF ACTION of the for
point. The perpendicular distance is called the LEVER-ARM, and
the CENTER OF MOMENTS. Forces acting upward are considered PO
those acting downward are considered NEGATIVE. The center of mo
be taken at any point in the plane of action of the forces, but
convenient to take it at one of the reactions. For example, the bear
supports a concentrated load P at the distance m from the left suppor
the left reaction take the center of moments at the right reaction.
EQUATION OF MOMENTS is

$$R_1 l - Pn = 0.$$

from which

$$R_1 = Pn/l$$

like manner, to find R_2 the center of moments is taken at R_1 and the
ation of moments is

$$R_2l - Pm = 0, \text{ from which } R_2 = Pm/l \qquad (1)'$$

n the first principle of statics mentioned, $R_1 + R_2$ must equal P; hence, as
eck, $Pn/l + Pm/l = P$.

ample 1. Let a beam 15 ft in span support a concentrated load of 700 lb,
from the left end; or, $P = 700$, $m = 6$ and $n = 9$. Then, from Formula (1),
$700 \times 9/15 = 420$ lb. $R_2 = 700 \times 6/15 = 280$ lb and $420 + 280 = 700$ lb.
a concentrated load at the middle, or for a uniform load over a simple
, it is evident without applying the conditions of equilibrium, that each
on is one-half the load, for, in Formulas (1) and (1)′, m and n each equal
nd R_1 and $R_2 = \frac{1}{2} P$.
any number of concentrated loads (Fig. 2) the reactions may be found by
together the reactions found by Formula (1) due to each load separately,
may be computed in one operation by the following formula:

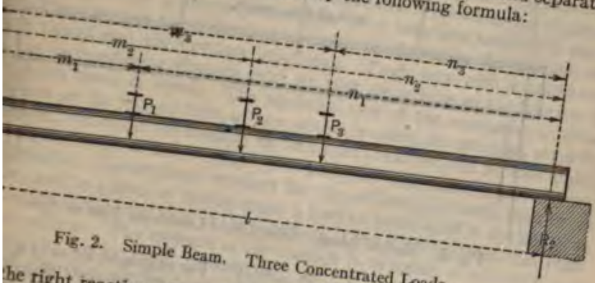

Fig. 2. Simple Beam. Three Concentrated Loads

he right reaction, the center of moments is taken at the left support,
uation of moments is

$$R_2l - P_1m_1 - P_2m_2 - P_3m_3 = 0$$

$$R_2 = \frac{P_1m_1 + P_2m_2 + P_3m_3}{l} \qquad (2)$$

nner, to find R_1 the center of moments is taken at R_2 and the equa-
ents is

$$R_1l - P_1n_1 - P_2n_2 - P_3n_3 = 0$$

$$R_1 = \frac{P_1n_1 + P_2n_2 + P_3n_3}{l} \qquad (3)$$

Suppose the beam in Fig. 2 is 20 ft in length. Let there be
ted loads of 500, 800 and 600 lb placed 5, 9 and 12 ft respectively
pport. Then $l = 20$, $m_1 = 5$, $m_2 = 9$, $m_3 = 12$, $P_1 = 500$, $P_2 = 800$
Substituting in Formulas (2) and (3),

$$R_2 = \frac{500 \times 5 + 800 \times 9 + 600 \times 12}{20} = 845 \text{ lb}$$

$$_1 = \frac{500 \times 15 + 800 \times 11 + 600 \times 8}{20} = 1\,055 \text{ lb}$$

$500 + 800 + 600 = 845 + 1\,055 = 1\,900$ lb

To find the reactions for a combination of uniformly distribute
centrated loads, to each of the reactions obtained by Formulas (1) or
concentrated loads, add one-half the distributed load. Thus,
20-ft beam in this example weighs 40 lb per linear ft. This is con
uniformly distributed load and for the entire beam it is 40 lb ×
By the rule, one-half of this is added to each reaction, so that the tot
are, $R_1 = 845 + 400 = 1\,245$ lb and $R_2 = 1\,055 + 400 = 1\,455$ lb.

Example 3. For a distributed load applied over only a part of the
Fig. 3, assume the load to be CONCENTRATED AT THE MIDDLE of th

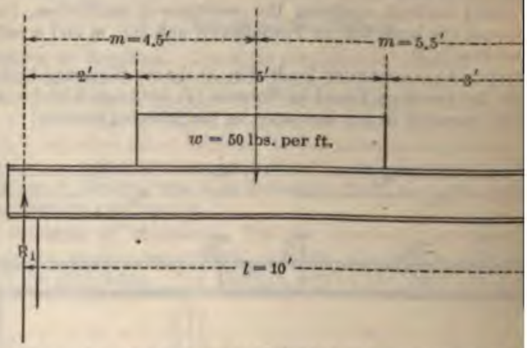

Fig. 3. Simple Beam. Distributed Load over Part of Span

which it acts and use Formulas (1) and (1)'. For example, let w (Fig
lb per linear ft, applied for a distance of 5 ft over the beam. Then
load, is 50 lb × 5 = 250 lb. This may be assumed to be concent
center, 4.5 ft from the left support. Then $P = 250$, $m = 4.5$ and
from Formulas (1) and (1)',

$$R_1 = \frac{250 \times 5.5}{10} = 137.5 \text{ lb}$$

and

$$R_2 = \frac{250 \times 4.5}{10} = 112.5 \text{ lb}$$

Therefore, for any combination of concentrated and uniform loads
over the entire beam, or over only part of it, find the reactions due
centrated loads by Formulas (1) or (2), and to them add the reaction
uniformly distributed loads.

2. Bending Moments in Beams*

Definitions. The bending moment is a measure of the tendencies
break a beam by BENDING or FLEXURE. Fig. 4 shows the manner
simple beam, supported at the ends, breaks when subjected to a l
than it can bear. The effect of a load upon a beam is to cause it
BEND. The bending of the beam shortens, or compresses, the upper
stretches, or elongates, the lower fibers. So long as the resistance o

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* See, also, Chapter XV, pages 555 to 563.

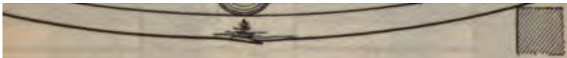


Fig. 4. Manner of Rupture of Simple Beam

to calculate the FLEXURAL STRENGTH OF A BEAM, it is necessary to
ne nature and extent, first, of the EXTERNAL FORCES acting to break
and secondly of the INTERNAL FORCES or STRESSES tending to resist
The external forces tending to break the beam by flexure are the
LOADS and the UPWARD REACTIONS. Each acts with a LEVERAGE
ne perpendicular distance from its LINE OF ACTION to the section at
beam tends to break. The algebraic sum of the moments of these
rces on the left, or right, of any section is called the BENDING MOMENT
tion, since it is the MOMENT OF THE RESULTANT OF THE FORCES which
nd the beam at that section. It is generally designated by M. Then,
efinition, the BENDING MOMENT for any section of a beam resting on
ts and in a state of flexure under a load or loads is M = the moment
action minus the sum of the moments of the loads between that reac-
ne section. The moment of the reaction is UPWARD, or POSITIVE, and
nt of any load DOWNWARD, or
if the part of the beam on the left
ion is considered.

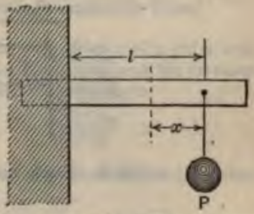


Fig. 5. Cantilever Beam. Concentrated Load near Free End

ng Moments in Beams for Dif-
ferent Kinds of Loading

Case I

xed at One End and Loaded with a
ed Load P, Near the Free End

m bending moment, at wall $= P \times l$
moment at any other section $x = Px$

Case III

Beam Fixed at One End and Loaded with Both a Concentrated and Distributed Load (Fig. 7).

Maximum bending moment, at wall $= P \times l_2 + W \times l_1/2$

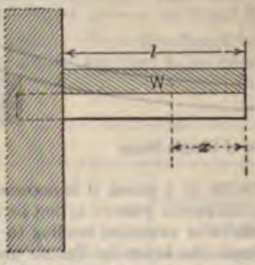


Fig. 6. Cantilever Beam. Uniformly Distributed Load

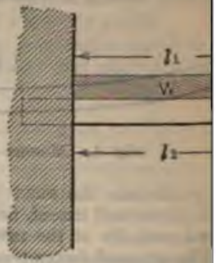


Fig. 7. Cantilever Beam. uted Load and Load at F

Case IV

Beam Supported at Both Ends and Loaded with a Concentrated Middle (Fig. 8).

Maximum bending moment, under the load $= Pl/4$

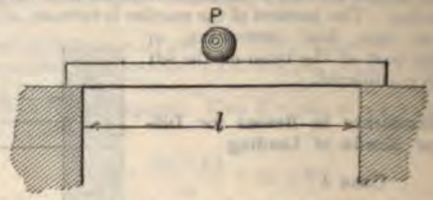


Fig. 8. Simple Beam. Concentrated Load at the Middle

Case V

Beam Supported at Both Ends and Loaded with a Uniformly Load W (Fig. 9).

Maximum bending moment, at the middle $= Wl/8$

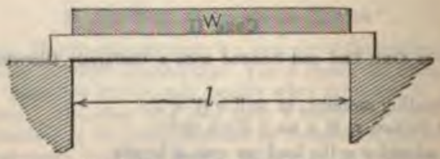


Fig. 9. Simple Beam. Uniformly Distributed Load

Case VI

upported at Both Ends and Loaded with a Concentrated Load not at the 'ig. 10).

um bending moment, under the load = Pmn/l

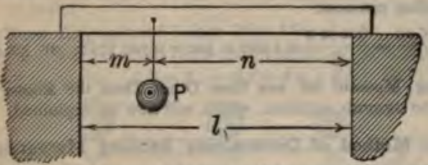
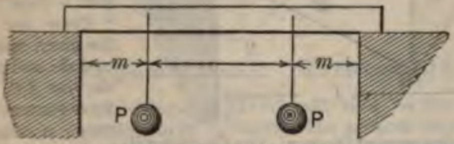


Fig. 10. Simple Beam. Concentrated Load not at the Middle

Case VII

Supported at Both Ends and Loaded Symmetrically with Two Equal ited Loads (Fig. 11).

um bending moment $= Pm$ and is the same for any section of the beam the two loads.



. 11. Simple Beam. Two Concentrated Loads Symmetrically Placed

these examples it will be seen that all the quantities which enter into utation of the bending moment are the load, the span and the distance int of application of the load from the center of moments.

Case VIII

Supported at Both Ends and Loaded with a Distributed Load Over Part an (Fig. 12).

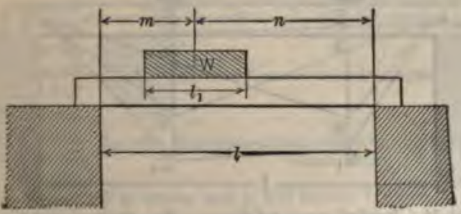


Fig. 12. Simple Beam. Distributed Load over Part of Span

um bending moment, under the center of the load $= Wmn/l - Wl_1/8$
and n are equal the bending moment $= W \times l/4 - W \times l_1/8$

Example 4. In Fig. 12 let $W = 800$ lb, $m = 8$ ft, $n = 12$ ft, $l = 20$ ft a
Then the bending moment

$$= \frac{800 \times 8 \times 12}{20} - \frac{800 \times 8}{8} = 3\,840 - 800 = 3\,040 \text{ lb, or } 36\,4$$

Example 5. In Fig. 12 let $m = n = 10$ ft, $l = 20$ ft. $l_1 = 4$ ft and
Then the bending moment

$$= \frac{600 \times 20}{4} - \frac{600 \times 4}{8} = 3\,000 - 300 = 2\,700 \text{ ft-lb, or } 32\,400$$

The Bending Moment for any Case Other Than the Above ma
obtained by the GRAPHIC METHOD, which will now be explained.

4. Graphic Method of Determining Bending Moments in

Beam with One Concentrated Load (Fig. 13).

The BENDING MOMENT of a beam supported at both ends and load
concentrated load may be determined GRAPHICALLY, as follows:

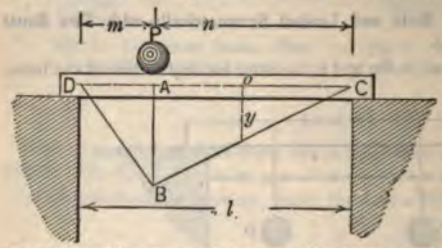


Fig. 13. Bending-moment Diagram. One Concentrated Load

Let P be
applied as sho
by the rule und
the MAXIMUM
MOMENT is und
and $= Pmn/l$
Draw the
the given span
to scale, and m
the line AB, t
FOOT-POUNDS to
INCH, a dist
to the bendin
Connect B wit
of the beam. To find the bending moment at any other point of
as at o, draw the vertical line y to BC. Its length, measured t
scale to which AB is drawn, will give the bending moment at o.
$DBCAD$ is called the BENDING-MOMENT DIAGRAM and the lines B
are called INFLUENCE LINES for the bending moments.

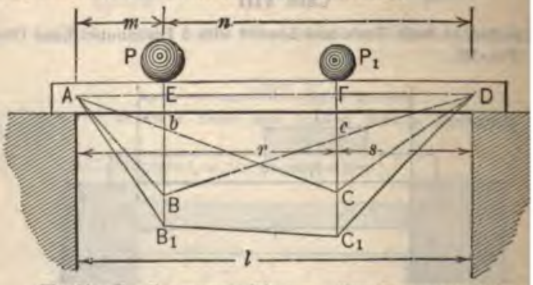


Fig. 14. Bending-moment Diagram. Two Concentrated Loads

Beam with Two Concentrated Loads (Fig. 14).

To draw the bending-moment diagram for a beam with two co
loads, draw the dotted lines ABD and ACD, giving the BENDING-M

each load separately. *EB* is laid out to scale, equal to Pmn/l and o P_1rs/l

ding moment at the point E is equal to EB (from the load P) $+ Eb$ load P_1), or $M = EB + Eb = EB_1$; and at F the bending moment is $FC + Fc = FC_1$. The BENDING-MOMENT DIAGRAM for both loads is ıd the MAXIMUM BENDING MOMENT is, in this particular case, the line ıred to scale.

ith Three Concentrated Loads (Fig. 15).

as in the last case, and draw the BENDING-MOMENT DIAGRAM for each arately. Make $AD = A_1 + A_2 + A_3$, $BE = B_1 + B_2 + B_3$ and

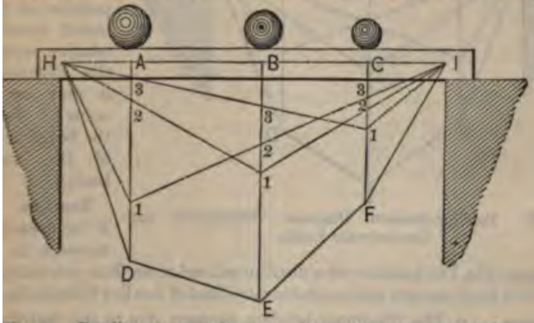


Fig. 15. Bending-moment Diagram. Three Concentrated Loads

$C_2 + C_3$. The figure *HDEFIH* will then be the BENDING-MO- RAM corresponding to all the loads. The BENDING-MOMENT DIAGRAM with any number of concentrated loads may be drawn in the same

ith a Uniformly Distributed Load (Fig. 16).

e beam with the given span, accurately to a scale as before, and at of the beam draw the vertical line AB, to a scale of a certain number OUNDS to AR INCH, $Wl/8$, from represent- ole distrib- Connect C, B, D by A to obtain G-MOMENT To find g moment nt a, draw al line ab, it to the to which AB is drawn, and it will be the bending moment desired. or drawing the PARABOLA will be found in Part I, page 79.

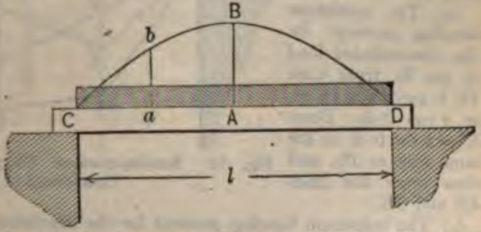


Fig. 16. Bending-moment Diagram. Distributed Load over Whole Beam

oaded with Both Distributed and Concentrated Loads (Fig. 17).

rmine the bending *moments in this case, combine the* BENDING-MOMENT *for the concentrated loads and for the distributed load,* as shown in

Fig. 17. The bending moment at any section of the beam will then b by the line *ABC* on top and by the line *CDEFA* on the bottom; and IMUM BENDING MOMENT will be the longest vertical line that can b between th bounding line

For exam bending mom is *BE*. The MAXIMUM BEN MENT depends position of th trated loads relative mag the distribut it may or occur at the the beam or of the con loads.

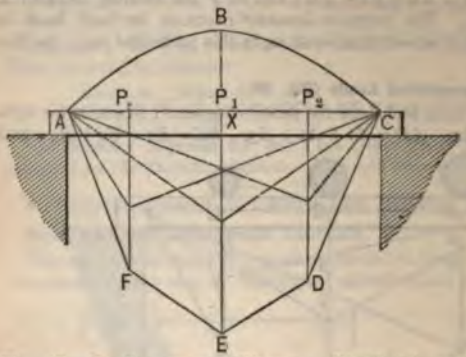


Fig. 17. Bending-moment Diagram. Distributed and Concentrated Loads

Example (is the greates moment in a 20 ft span (Fig. 18), loaded with a distributed load of 800 lb, a concentra 500 lb 6 ft from one end, and a concentrated load of 600 lb 7 ft from the

Solution. (1) The maximum bending moment due to the distribu from Case V, is $Wl/8$, or $800 \times 20/8 = 2\,000$ ft-lb. Lay off vertically middle of the beam, and at any convenient scale, say 4 000 ft-lb to the inch, $B1 = 2\,000$ ft-lb, and draw a parabola through the points *A*, *B* and *C*. (See page 79.)

(2) The maximum bending moment for the concentrated load of 500 lb, from Case VI, is $500 \times 6 \times 14/20$, or 2 100 ft-lb. Draw $E2 = 2\,100$ ft-lb to the same scale as $B1$, and then draw the lines *AE* and *CE*.

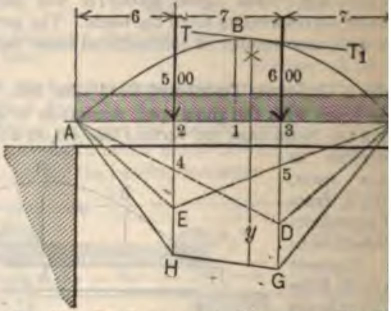


Fig. 18. Bending-moment Diagram. Distrib Concentrated Loads

(3) The maximum bending moment for the concentrated load of like manner, is $600 \times 7 \times 13/20$, or 2 730 ft-lb. Draw $D3 = 2\,730$ connect *D* with *A* and *C*.

(4) Make *EH* equal to the distance from 2 to 4, and *DG* to the dist 3 to 5, and draw *AHGC*.

The MAXIMUM BENDING MOMENT will be represented by the longes line which can be drawn between the parabola *ABC* and the broken lin In this example the longest vertical line which can be drawn is X scales 5 550 ft-lb.

position of the line Xy is determined by drawing the line TT_1 parallel and tangent to ABC. The vertical line Xy is drawn through the point ency.

. To change the bending moment to INCH-POUNDS multiply the mo- FOOT-POUNDS by 12.

MODULUS AND RADIUS OF GYRATION

By

CHARLES P. WARREN

ASSISTANT PROFESSOR OF ARCHITECTURE, COLUMBIA UNIVERSI

1. The Properties of Cross-Sections

The Moment of Inertia. The strength of a cross-section to resis
in either a beam or a column, depends not only upon the area but
the form of the cross-section. The parts of the cross-section farthest
neutral axis, which always passes through the center of gravity of
section, are much more efficient in resisting bending stresses than th
adjacent to the axis; so that some mathematical expression must be
that will represent the efficiency of the entire cross-section to resis
stresses when compared with that of any other cross-section. This ex
called the MOMENT OF INERTIA and is usually designated by the letter I.

The Moment of Inertia of any cross-section may be defined as t
the products obtained by multiplying each of the elementary areas of
section is composed by the square of its normal distance from the ne
of the section.

By an ELEMENTARY AREA is meant an area smaller than any deal
simple mathematics, and it is, therefore, impossible to find an exact
for the moment of inertia of a cross-section by such methods. By me
calculus, however, exact formulas have been deduced from which the
of inertia of simple geometrical forms, such as rectangles, triangles, ci
may be found, with respect to different axes.

The NEUTRAL AXIS of the cross-section of a beam, girder, column, e
is in a state of flexure, is the line on which there is neither tension nor
sion in the fibers, and when the unit stresses do not exceed the ELASTI
the material, it can be shown that this neutral axis passes through the c
GRAVITY of the cross-section. The normal distance of the extreme fiber
neutral axis is usually designated by the letter *c* or the letter *y*. The
used in the notation of this book.

is the algebraic sum of all the moments of the internal horizontal that section with reference to a point in that section. It is usually d by the expression SI/c, in which S is the horizontal unit stress, compressive, as the case may be, upon the fiber most remote from the is of the section, and called the FIBER-STRESS; I is the MOMENT OF f the area of the section with reference to the NEUTRAL AXIS; and c rtest distance from the most remote fiber to that axis. Since, for n of forces and stresses at any cross-section of a beam, the bending quals the resisting moment for that section, if M represents the bend- nt we have the equation

$$M = SI/c \qquad (1)$$

known as the FLEXURE FORMULA and is universally used for investi- flexural strength of beams.

ction-Modulus or Section-Factor. That expression I/c in the above s generally known as the SECTION-MODULUS or SECTION-FACTOR. tity for the principal rolled sections is given in Tables IV, V, VI, VII, XII, XIII and XIV. Corresponding to the two moments of inertia used for all sections (except for squares and circles) there are two oduli also, one for each axis. Thus, the section-modulus of the 12-in beam, with respect to a neutral axis perpendicular to the web, is 8/6 = 36; and for the axis parallel to the web, it is $I/c = 9.5/2.5 = 3.8$. shape the section-modulus may be found by dividing the moment of the normal distance of the extreme fiber from the neutral axis.

adius of Gyration. The effect of the form of the cross-section of a its strength is determined by a quantity called the RADIUS OF GYRA- ch is as necessary in the determination of the strength of a column as nt of inertia is in the determination of the strength of a beam It is y the letter r. The value of the radius of gyration for any section ined by the formula

$$r = \sqrt{I/A} \qquad (2)$$

shapes are given in the tables of the properties of sections, mentione For the 12-in 31.5-lb **I** beam, $r = 4.83$ in and $r' = 1.01$ in. The radius of of any other section may be found by Formula (2).

Formulas for the moments of inertia, radii of gyration and sectio of the principal elementary sections are given on the following pages case of a hollow section or a section with a reentering hollow part, th of inertia of the hollow part is to be subtracted from that of the enclo Moments of inertia when referred to the same axis can be added or s like any other quantities which are of the same kind.

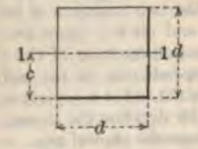
2. Areas, Moments of Inertia, Section-Moduli and Radi Gyration of Elementary Sections

I = the moment of inertia
I/c = the section-modulus
r = the radius of gyration
A = the area of the section
c = the normal distance of most remote fiber from neutra

The position of axis referred to in each case is represented by the l $1-1$.

SQUARE

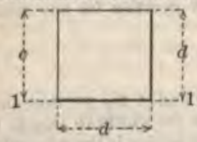
Axis of moments through center



$$A = d^2$$
$$c = \frac{d}{2}$$
$$I = \frac{d^4}{12}$$
$$\frac{I}{c} = \frac{d^3}{6}$$
$$r = \frac{d}{\sqrt{12}} = 0.288675\,d$$

SQUARE

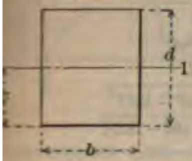
Axis of moments on base



$$A = d^2$$
$$c = d$$
$$I = \frac{d^4}{3}$$
$$\frac{I}{c} = \frac{d^3}{3}$$
$$r = \frac{d}{\sqrt{3}} = 0.577350\,d$$

RECTANGLE

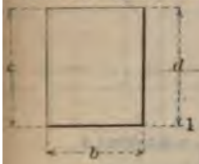
of moments through center



$$A = bd$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3}{12}$$
$$\frac{I}{c} = \frac{bd^2}{6}$$
$$r = \frac{d}{\sqrt{12}} = 0.288675\,d$$

RECTANGLE

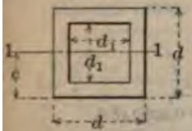
s of moments on base



$$A = bd$$
$$c = d$$
$$I = \frac{bd^3}{3}$$
$$\frac{I}{c} = \frac{bd^2}{3}$$
$$r = \frac{d}{\sqrt{3}} = 0.577350\,d$$

HOLLOW SQUARE

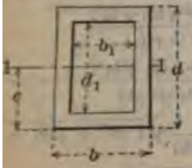
is of moments through center



$$A = d^2 - d_1^2$$
$$c = \frac{d}{2}$$
$$I = \frac{d^4 - d_1^4}{12}$$
$$\frac{I}{c} = \frac{d^4 - d_1^4}{6\,d}$$
$$r = \sqrt{\frac{d^2 + d_1^2}{12}}$$

LLOW RECTANGLE

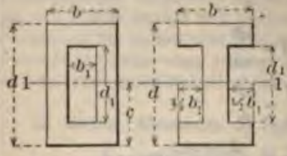
s of moments through center



$$A = bd - b_1d_1$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3 - b_1d_1^3}{12}$$
$$\frac{I}{c} = \frac{bd^3 - b_1d_1^3}{6\,d}$$
$$r = \sqrt{\frac{bd^3 - b_1d_1^3}{12\,(bd - b_1d_1)}}$$

HOLLOW RECTANGLE AND I BEAM

Axis of moments through center



$$A = bd - b_1d_1$$

$$c = \frac{d}{2}$$

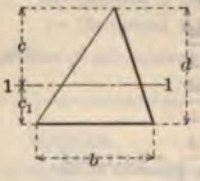
$$I = \frac{bd^3 - b_1d_1^3}{12}$$

$$\frac{I}{c} = \frac{bd^3 - b_1d_1^3}{6\,d}$$

$$r = \sqrt{\frac{bd^3 - b_1d_1^3}{12\,(bd - b_1d_1)}}$$

TRIANGLE

Axis of moments through center of gravity



$$A = \frac{bd}{2}$$

$$c = \frac{2\,d}{3} \qquad c_1 = \frac{d}{3}$$

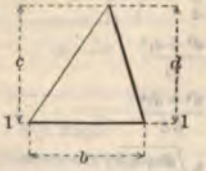
$$I = \frac{bd^3}{36}$$

$$\frac{I}{c} = \frac{bd^2}{24}$$

$$r = \frac{d}{\sqrt{18}} = 0.235702\,d$$

TRIANGLE

Axis of moments on base



$$A = \frac{bd}{2}$$

$$c = d$$

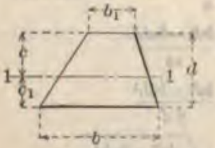
$$I = \frac{bd^3}{12}$$

$$\frac{I}{c} = \frac{bd^2}{12}$$

$$r = \frac{d}{\sqrt{6}} = 0.408248\,d$$

TRAPEZOID

Axis of moments through center of gravity



$$A = \frac{d\,(b + b_1)}{2}$$

$$c^* = \frac{d\,(b_1 + 2\,b)}{3\,(b + b_1)} \qquad {}^*c_1 = \frac{d\,(}{3}$$

$$I = \frac{d^3\,(b^2 + 4\,bb_1 + b_1^2)}{36\,(b + b_1)}$$

$$\frac{I}{c} = \frac{d^2\,(b^2 + 4\,bb_1 + b_1^2)}{12\,(b_1 + 2b)}$$

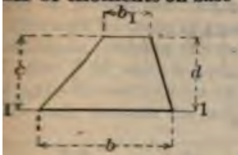
$$r = \frac{d}{6\,(b + b_1)}\sqrt{2\,(b^2 + 4b}$$

---

* To find c and c_1, see Chapter VI, page 295.

TRAPEZOID

xis of moments on base



$$A = \frac{d\,(b+b_1)}{2}$$

$$c = d$$

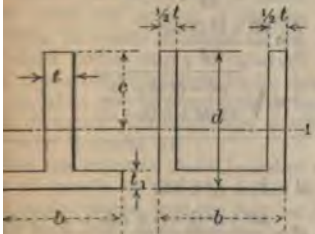
$$I = \frac{d^3\,(b+3\,b_1)}{12}$$

$$\frac{I}{c} = \frac{d^2\,(b+3\,b_1)}{12}$$

$$r = \frac{d}{\sqrt{6}}\sqrt{\frac{b+3\,b_1}{b+b_1}}$$

SECTION AND CHANNEL

s of moments through center of gravity



$$A = td + t_1\,(b-t)$$

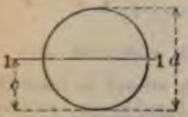
$$c^* = \frac{td \times ½\,d + t_1\,(b-t)\,(d - ½\,t_1)}{A}$$

$$I = \frac{tc^3 + bc_1^3 - (b-t)\,(c_1 - t_1)^3}{3}$$

$$r = \sqrt{\frac{I}{A}}$$

CIRCLE

Axis of moments through center



$$A = \frac{\pi d^2}{4} = 0.785398\,d^2$$

$$c = \frac{d}{2}$$

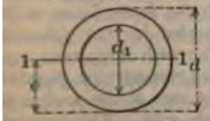
$$I = \frac{\pi d^4}{64} = 0.049087\,d^4$$

$$\frac{I}{c} = \frac{\pi d^3}{32} = 0.098175\,d^3$$

$$r = \frac{d}{4}$$

HOLLOW CIRCLE

Axis of moments through center



$$A = \frac{\pi\,(d^2 - d_1^2)}{4} = 0.785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi\,(d^4 - d_1^4)}{64} = 0.049087\,(d^4 - d_1^4)$$

$$\frac{I}{c} = \frac{\pi\,(d^4 - d_1^4)}{32\,d} = 0.098175\,\frac{(d^4 - d_1^4)}{d}$$

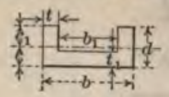
$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

---

* *To find the values of c and c_1, see Chapter VI, page 295.*

CHANNEL

Axis of moments through center of gravity



$$A = t_1b + 2\,t\,(d - t_1)$$

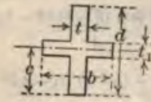
$$c^* = \frac{2\,d^2t + d_1t_1^2}{2\,A}$$

$$I = \frac{2\,td^3 + b_1t_1^3}{3} - Ac^2$$

$$r = \sqrt{\frac{I}{A}}$$

CROSS-SECTION

Axis of moments through center of gravity



$$A = td + t_1\,(b - t)$$

$$c = \frac{d}{2}$$

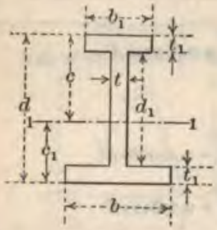
$$I = \frac{td^3 + t_1^3\,(b - t)}{12}$$

$$\frac{I}{c} = \frac{td^3 + t_1^3\,(b - t)}{6\,d}$$

$$r = \sqrt{\frac{td^3 - t_1^3\,(b - t)}{12\,(td + t_1\,(b - t))}}$$

IRREGULAR I SHAPE

Axis of moments through center of gravity



$$A = bt_1 + d_1t + b_1t_1$$

$$c^* = \frac{td^2 + t_1^2\,(b - t) + t\,(b_1 - t)\,(}{2\,A}$$

$$I = \frac{b_1c^3 - (b_1 - t) \times (c - t_1)^3}{3}$$

$$+ \frac{bc_1^3 - (b - t) \times (c_1 -}{3}$$

$$r = \sqrt{\frac{I}{A}}$$

---

* To find c and c_1, see Chapter VI, page 295.

3. Transferring Moments of Inertia to Other Parallel A

Explanation of Formula. It is often necessary to determine the
of inertia with respect to some other axis than the one passing thr
center of gravity of the section, such, for example, as one passing thr
base and parallel to the other. Suppose it is desired to find the mom
ertia of a rectangle about an axis passing through the lower base,
second figure on page 335. It may be demonstrated by the pri
mechanics that the moment of inertia of any section with respect to
is equal to the moment of inertia of the section with respect to a pa
through the center of gravity, plus the product of the area of the secti
plied by the square of the normal distance between the axes. This ru
expressed by the formula

$$I_1 = I + Ak^2$$

$_1$ is the required moment of inertia, I the moment of inertia of the :h respect to the axis through its center of gravity and parallel to the A the area of the section and h the normal distance between the axes. it is seen that the moment of inertia of any section-area is less for rough its center of gravity than for any other parallel axis.

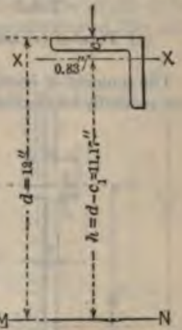


Fig. 1. Moment of Inertia of Cross-section of Steel Angle

mple, consider the rectangle shown on page 335, of breadth b and he I of which is known to be $bd^3/12$ for an axis passing through the ravity and parallel to the base. Then, for axis through the base, the above formula

$$= \frac{bd^3}{12} + bd \times \left(\frac{d}{2}\right)^2 = \frac{bd^3}{12} + \frac{bd^3}{4} = \frac{bd^3}{3}$$

ne moment of inertia of the cross-section el angle shown in Fig. 1, about the axis qual to the moment of inertia about the plus the product of its area multiplied he moments of inertia for the sections of ard rolled shapes of structural steel may from the tables given in this chapter. The , also, may be found from the same tables; distance subtracted from d will give the of Formula (3).

, for example, that it is desired to find the of inertia of the cross-section of a 4 by 3 ngle, placed, with the long leg horizontal, axis MN, 12 in from the back (Fig. 1). Turning to Table XI, the e angle-section = 3.25 sq in. I, the moment of inertia of the angle-out an axis 2–2, or XX of Fig. 1, parallel to the long leg = 2.4, c_1, the of this axis from the back of the long leg = 0.83 in and h, the distance he axes = $(d - c_1) = 12 - 0.83$ in = 11.17 in. Substituting these values la (3)

$$I_1 = 2.4 + 3.25 \times 11.17^2 = 2.4 + 405.50 = 407.9$$

4. Moments of Inertia of Compound Sections

Ioment of Inertia of a Compound Section made up of a number of ctions may be found by the same formula, $I_1 = I + Ah^2$. Denote the HE MOMENTS OF INERTIA of the separate sections making up the com-ction, with respect to an axis through the center of gravity of that y ΣI_1. Formula (3) then becomes

$$\Sigma I_1 = \Sigma\,(I + Ah^2) \qquad (4)$$

, to find the moment of inertia of any compound section made up of of smaller sections:

d the moment of inertia of each of the smaller sections about an axis rough its own center of gravity and parallel to the neutral axis of the l section;

ltiply the area of each of the smaller sections by the square of the dis-ween its center of gravity and the center of gravity of the *whole figure;*

d the results found by (1) *and (2) for the moment of inertia of the* ure.

mple, consider the cast-iron beam or lintel shown in section in Fig. 2:

(1) I of upper flange-section $= 4 \times 1^3/12 = 4/12$
I of web-section $= 1 \times 18^3/12 = 5\,832/12$
I of lower flange-section $= 16 \times 1^3/12 = 16/12$
Total $= 5\,852/12 = 487.6$

(2) Ah^2 for the upper flange $= 4 \times (12.5)^2 = 625$
Ah^2 for the web $= 18 \times 3^2 = 162$
Ah^2 for the lower flange $= 16 \times (6.5)^2 = 676$
Total $= 1\,463$

(3) Total of (1) and (2) $= 487.6 + 1\,463 = I_1$ of compound section $= 1$

The moment of inertia of the cross-section of any compound beam, can generally be readily found by using the tables of properties of sectio

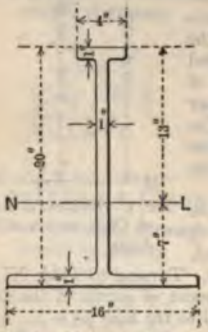


Fig. 2. Moment of Inertia of Cross-section of Cast-iron Lintel

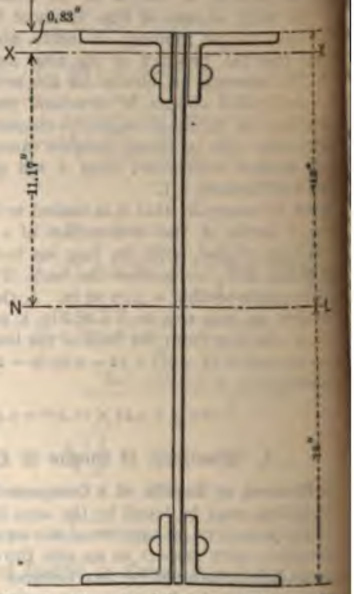


Fig. 3. Moment of Inertia of Cross-sectio of Plate Girder. No Flange-plates

give the numerical values of I for the various rolled shapes of which the beam is composed, with respect to the axis through the center of gravity.

The Moment of Inertia of a Single-Web Girder-Section. Consider, for example, the single-web girder shown in section in Fig. 3, and made up of one ½ by 24-in web and four 4 by 3 by ½-in flange-angles with the long legs placed horizontally. T ing to Table XI, the moment of inertia of the cross-section of one these angles about an axis XX (2–2 in the table) parallel to the leg = 2.4, and the distance of this axis from the back of the long leg (y in t table) = 0.83 in; hence h, the distance between the axis of the angle-sec and the axis of the girder-section $= 12 - 0.83 = 11.17$ in. A, from the = 3.25 sq in. The moment of inertia of the cross-section of each angle the axis of the girder, therefore, from Formula (3), is $I_1 = 2.4 + 3.25 \times (11.17$ 407.9, and for the four angles = 1631.6. Since the axis of the cross-sectio

is coincident with the axis of the section of the girder, its
inertia = $bd^3/12 = \frac{1}{2} \times (24)^3/12 = 576$. This may be found
Table I, page 346, Moments of Inertia of Rectangles. The
ertia, therefore, of the section of the compound girder = 1631.6
.6.

**ent of Inertia of a Section of a Compound Girder with
s** is found in the same way, except that the moments of inertia of
f the flange-plates with respect to the axis of the girder-section
d to the moments of inertia of the cross-sections of the other
he girder in Fig. 4 is com-
30 by ⅜-in web-plate, four
-in angles, with the longer
al, and two 12 by ½-in

cross-section of web (from
I, page 346) = 843.75
h angle-section = $I + Ah^2$
ula 3)

XI, for each flange-angle,
.75 and the perpendicular
center of gravity to back
1.10 in. Hence $h = 15 -$
in. $I_1 = 6.6 + 4.75 \times$
.35; and for four angles
for the cross-section of
ate = $12 \times (\frac{1}{2})^3/12 = 0.125$,
6 sq in and $h = 15 + \frac{1}{4}$
For each flange-plate, then,
$+ 6 \times (15.25)^2 = 1395.125$;
wo plates, 2 790.25. The
ertia for the cross-section
girder, therefore, with
he horizontal axis passing
center of gravity of the
75 + 3 697.4 + 2 790.25 =

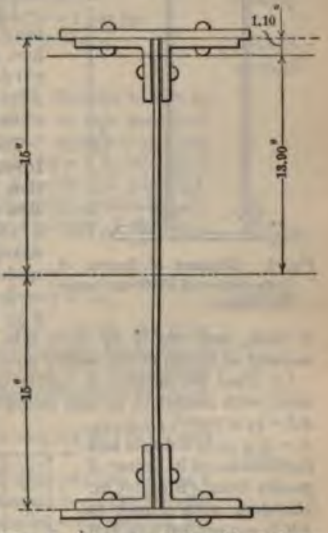


Fig. 4. Moment of Inertia of Cross-section of Plate Girder with Flange-plates

noticed that the moments
the cross-sections of the flange-plates and angles about their own
is so small, compared with their moments of inertia about the
of the girder-section, that they might be omitted without any
rror. Therefore, in calculating the moments of inertia for riveted
he custom of many engineers to let $I_1 = Ah^2$ for flange-plate and
. In that case, for the girder-section in Fig. 4,

I for web = 843.75
I_1 for angles = Ah^2 = 3 671.00
I_1 for flange-plates = Ah^2 = 2 790.00
Moment of inertia of entire girder-section = 7 304.75

ent of Inertia of a Section ***of a Box Girder.*** *Let the box
in Fig. 5 be composed of two ⅜ by 30-in webs, two 16 by ½-in
nd four 4 by 3 by ½-in angles with the long legs horizontal.*

I for each flange-plate $= bd^3/12 = 16 \times (½)^3/12 = 0.16$; $A = ½ \times 16$ in and $h = 15 + ¼ = 15.25$ in. $I_1 = I + Ah^2 = 0.16 + 8 \times (15.25)^2 = 1\,8$ for the two flange-plates, 3 721.28. I angle = 2.4, A = 3.25 and the distance back of the long leg to an axis th center of gravity of the angle, paral long leg = 0.83 in; so that $h = 15 - 0.8$ in. I_1 for the four angles is $(4 \times 2$ $3.25) \times (14.17)^2 = 2\,619$. I for each I, page 346) = 843.75 and for the = 1 687.5. The moment of inertia, therefore, for the entire girder-section = 3 721.28 + 2 619 + 1 687.5 = 8 027.78.

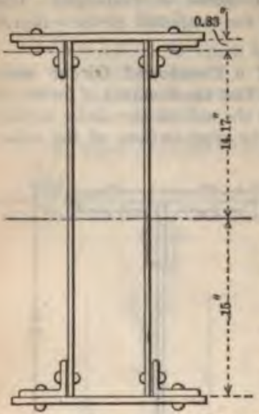


Fig. 5. Moment of Inertia of Cross-section of Plate-and-angle Box Girder

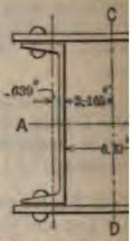


Fig. 6. Momen of Cross-sectio and-channel B

The Moment of Inertia of the Section of a Channel Box Column. Fig. 6 shows the cross-section of a column made up of two 10-in 15-lb channels, set 6.33 in apart, back to back, and two ½ by 12-in side plates. Let it be required to moment of inertia of the section about the two axes AB and CD.

(1) Find the moment of inertia about the axis AB. I, for one o plates with respect to an axis through its own center of gravity and $AB = 12 \times (½)^3/12 = 0.125$, $A = ½ \times 12$-in = 6 sq in and the distance of its center of gravity from AB is 5.25 in. Therefore, with respect to AB, $I_1 = 0.125 + 6 \times (5.25)^2 = 165.5$. The moment of inertia of a 10-in 15-lb channel with respect to an axis through its center of gravity and perpendicular to the web (Table VIII, page 359) = 66.9. Hence the moment of inertia of the whole column-section with respect to the axis AB = $(2 \times 165.5) + (2 \times 66.9) = 464.8$.

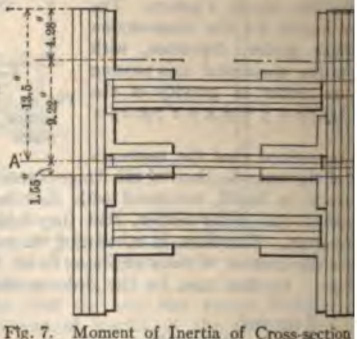


Fig. 7. Moment of Inertia of Cross-section web Plate-and-angle Box Column

(2) Find the moment of inertia about the axis CD. I, for one of the side plates (Table 346) = 72. I, for one of the channels with respect to an axis paral web = 2.30, A = 4.46 and the distance of the center of gravity from of the web = 0.64 in, approximately. Hence $h = 3.165 + 0.64$ $I_1 = 2.30 + 4.46 \times (3.8)^2 = 66.7$ and the moment of inertia of the wh section with respect to the axis CD = $(2 \times 72) + (2 \times 66.7) = 277.$

t of Inertia of the Section of a Heavy Plate-and-Angle 7 shows the cross-section of one of the basement-columns in ust Company Building, New York City. It is made up of six ch 27 by $\frac{3}{4}$ in section; two flange-plates, each 27 by $\frac{11}{16}$ in; es, each 6 by 6 by $\frac{15}{16}$ in; eight outer web-plates, each 18 by -angles, each 6 by $3\frac{1}{2}$ by $\frac{15}{16}$ in; and two middle web-plates n. What is its moment of inertia of the entire column-section the axis AB?

7 by $\frac{3}{4}$-in flange-plate (Table I) = 1 230.19
by $\frac{3}{4}$-in flange-plates = 1 230.19 × 6 = 7 381.14
7 by $\frac{11}{16}$-in web-plate (Table I) = 1 127.67
by $\frac{11}{16}$-in web-plates = 1 127.67 × 2 = 2 255.34
langes 9 636.48

-angles (Table XII, page 366) the area of a 6 by le = 10.37, its I with respect to an axis parallel to d passing through its center of gravity = 33.7 and this axis from the back of the leg = 1.84 in. Its I_1 the axis AB is found by Formula (3), page 338, = 13.5 − (0.12 + 4.16) = the distance from the axis lel axis through the center of gravity of the angle ce, substituting in Formula (3),

$= 33.7 + 10.37 \times (9.22)^2 = 915.15$
for the four flange-angles = 915.15 × 4 = 3 660.60

eb is $4 \times \frac{11}{16}$ in = $2\frac{3}{4}$ in thick. Hence the I for about the horizontal axis through its center of $(2.75)^3/12 = 31.2$. A = 18 by 2.75 in = 49.5 sq in. m its center of gravity to the axis AB is 13.5 − (1.38 - 0.12) = 6.01 or, say 6 in.
a (3), therefore, $I_1 = 31.2 + 49.5 \times 6^2 = 1\,813.2$ and ebs $I_1 = 1\,813.2 \times 2 =$ 3 626.4
web-angles, from Table XI, page 363, the area of a -in angle = 8.03, its I with respect to an axis through vity and parallel to the long leg = 6.9 and the dis- is from the back of the long leg = 0.99 in. h, the nce between the two axes = $\frac{9}{16}$ in, or 0.5625 in f one of the middle web-plates) + 0.99 = 1.55 in, Therefore, for one web-angle, from Formula (3),

$$I_1 = 6.9 + 8.03 \times (1.55)^2 = 26.17$$

angles, $I_1 = 26.17 \times 4 =$ 104.68

veb-plates are together $\frac{9}{16}$ in × 2 = $1\frac{1}{8}$ in = 1.125 $(= I_1)$ for the two plates is $18 \times (1.125)^3/12 =$ 1.90
of inertia of the entire column-section for the axis the sum of these moments of inertia for the differ-

ght flange-plates	9 636.48
our flange-angles	3 660.60
ght outer web-plates	3 626.40
ur web-angles	*104.68*
iddle *web-plate*	*1.90*
of inertia for the entire section	***17 030.06***

12-in plates, or 12 sq in) + (the sum of the areas of the two channe sq in) = 20.92 sq in. I about $AB = 464.8$ and about $CD = 277.4$.

Therefore, r, with respect to the axis $AB = \sqrt{\frac{464.8}{20.92}} = 4.71$

and r_1, with respect to the axis $CD = \sqrt{\frac{277.4}{20.92}} = 3.68$

Since r_1 is the smaller, it is the value to be used in the column-form to be noted that this value of r agrees with the r of the 10-in channel- Table XXV, on page 533. The value of r_1 does not, however, agree ex the r_1 in the same table, the variation being caused by a difference in t of the channels, back to back.

The Least Radius of Gyration of a Section of a Plate-a Column. As another example, let it be required to find the least gyration of the cross-section of the angle column shown in Fig. 8, made $\frac{3}{8}$ by 12-in web-plate, two $\frac{3}{8}$ by 12-in and 4 by 4 by $\frac{1}{2}$-in angles.

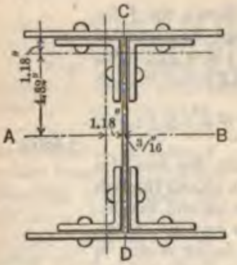


Fig. 8. Moment of Inertia of Cross-section of Plate-and-angle Column

(1) Find the moment of inertia abou AB. For the axis AB, I for each one plates with respect to an axis throug center of gravity and parallel to the a $12 \times (\frac{3}{8})^3/12 = 0.05$. $A = \frac{3}{8} \times 12 = 4.5$ $h = 6\frac{3}{16}$ in. $I_1 = 0.05 + 4.5 \times (6\frac{3}{16})^2$ I for each one of the angles with res axis through its center of gravity an with the flange-leg is 5.6, $A = 3.75$ an tance of the center of gravity from th the flange of the angle $= 1.18$. Hence $- 1.18$ in $= 4.82$ in and I_1 for each a $+ 3.75 \times (4.82)^2 = 92.71$. I for the we

adius of Gyration of the Cross-Section of a Hollow Rectangular
As another example, let it be required to find the radius of gyration
ss-section of a hollow rectangular cast-iron column with outside dimen-
6 in and with a shell ¼ in thick. (See figures and formulas for hol-
es and rectangles, page 335.) $A = 6^2 - (5.5)^2 = 36 - 30.25 = 5.75$ sq in.
$b_1b_1^3)/12 = [6^4 - (5.5)^4]/12 = (1\,296 - 910)/12 = 386/12 = 32.2$. $r^2 =$
$= 5.6$ and $r = 2.37$ in.

dii of gyration of round-section columns and square-section columns,
om 2 to 20 in in diameter and of metal varying from ¼ to 2 in thick,
in Tables II and III, see pages 348 to 351. For example: the radius
n of a 6 by 6-in square-section cast-iron column with a shell ¼ in
rom Table III, 2.35 in.

Table I.* Moments of Inertia of Rectangles

Neutral axis through center and normal to depth

Depth in inches	Widths of rectangles in inches					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
2	0.17	0.21	0.25	0.29	0.33	0.38
3	0.56	0.70	0.84	0.98	1.13	1.27
4	1.33	1.67	2.00	2.33	2.67	3.00
5	2.60	3.26	3.91	4.56	5.21	5.86
6	4.50	5.63	6.75	7.88	9.00	10.13
7	7.15	8.93	10.72	12.51	14.29	16.08
8	10.67	13.33	16.00	18.67	21.33	24.00
9	15.19	18.98	22.78	26.58	30.38	34.17
10	20.83	26.04	31.25	36.46	41.67	46.87
11	27.73	34.66	41.59	48.53	55.46	62.39
12	36.00	45.00	54.00	63.00	72.00	81.00
13	45.77	57.21	68.66	80.10	91.54	102.98
14	57.17	71.46	85.75	100.04	114.33	128.63
15	70.31	87.89	105.47	123.05	140.63	158.20
16	85.33	106.67	128.00	149.33	170.67	192.00
17	102.35	127.94	153.53	179.12	204.71	230.30
18	121.50	151.88	182.25	212.63	243.00	273.38
19	142.90	178.62	214.34	250.07	285.79	321.52
20	166.67	208.33	250.00	291.67	333.33	375.00
21	192.94	241.17	289.41	337.64	385.88	434.11
22	221.83	277.29	332.75	388.21	443.67	499.13
23	253.48	316.85	380.22	443.59	506.96	570.33
24	288.00	360.00	432.00	504.00	576.00	648.00
25	325.52	406.90	488.28	569.66	651.04	732.42
26	366.17	457.71	549.25	640.79	732.33	823.88
27	410.06	512.58	615.09	717.61	820.13	922.64
28	457.33	571.67	686.00	800.33	914.67	1029.00
29	508.10	635.13	762.16	889.18	1016.21	1143.23
30	562.50	703.13	843.75	984.38	1125.00	1265.63
32	682.67	853.33	1024.00	1194.67	1365.33	1536.00
34	818.83	1023.54	1228.25	1432.96	1637.67	1842.38
36	972.00	1215.00	1458.00	1701.00	1944.00	2187.00
38	1143.17	1428.96	1714.75	2000.54	2286.33	2572.13
40	1333.33	1666.67	2000.00	2333.33	2666.67	3000.00
42	1543.50	1929.38	2315.25	2701.13	3087.00	3472.88
44	1774.67	2218.33	2662.00	3105.67	3549.33	3993.00
46	2027.83	2534.79	3041.75	3548.71	4055.67	4562.63
48	2304.00	2880.00	3456.00	4032.00	4608.00	5184.00
50	2604.17	3255.21	3906.25	4557.29	5208.33	5859.38
52	2929.33	3661.67	4394.00	5126.33	5858.67	6591.00
54	3280.50	4100.63	4920.75	5740.88	6561.00	7381.13
56	3658.67	4573.33	5488.00	6402.67	7317.33	8232.00
58	4064.83	5081.04	6097.25	7113.46	8129.67	9145.87
60	4500.00	5625.00	6750.00	7875.00	9000.00	10125.00

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* This table may be used in computing the moments of inertia of plate columns and other compound sections in which plates are used. See pages 342.

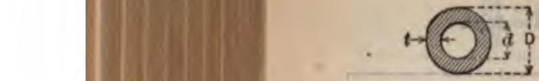
Table I* (Continued). Moments of Inertia of Rectangles

Neutral axis through center and normal to depth

Widths of rectangles in inches					
11/16	3/4	13/16	7/8	15/16	1
0.46	0.50	0.54	0.58	0.63	0.67
1.55	1.69	1.83	1.97	2.11	2.25
3.67	4.00	4.33	4.67	5.00	5.33
7.16	7.81	8.46	9.11	9.77	10.42
12.38	13.50	14.63	15.75	16.88	18.00
19.65	21.44	23.22	25.01	26.80	28.58
29.33	32.00	34.67	37.33	40.00	42.67
41.77	45.56	49.36	53.16	56.95	60.75
57.29	62.50	67.71	72.92	78.13	83.33
76.26	83.19	90.12	97.05	103.98	110.92
99.00	108.00	117.00	126.00	135.00	144.00
125.87	137.31	148.75	160.20	171.64	183.08
157.21	171.50	185.79	200.08	214.38	228.67
193.36	210.94	228.52	246.09	263.67	281.25
234.67	256.00	277.33	298.67	320.00	341.33
281.47	307.06	332.65	358.24	383.83	409.42
334.13	364.50	394.88	425.25	455.63	486.00
392.96	428.69	464.41	500.14	535.86	571.58
458.33	500.00	541.67	583.33	625.00	666.67
530.58	578.81	627.05	675.28	723.52	771.75
610.04	665.50	720.96	776.42	831.87	887.33
697.07	760.44	823.81	887.18	950.55	1013.92
792.00	864.00	936.00	1008.00	1080.00	1152.00
895.18	976.56	1057.94	1139.32	1220.70	1302.08
1006.96	1098.50	1190.04	1281.58	1373.13	1464.67
1127.67	1230.19	1332.70	1435.22	1537.73	1640.25
1257.67	1372.00	1486.33	1600.67	1715.00	1829.33
1397.29	1524.31	1651.34	1778.36	1905.39	2032.42
1546.88	1687.50	1828.13	1968.75	2109.38	2250.00
1877.33	2048.00	2218.67	2389.33	2560.00	2730.67
2251.79	2456.50	2661.21	2865.92	3070.63	3275.33
2673.00	2916.00	3159.00	3402.00	3645.00	3888.00
3143.71	3429.50	3715.29	4001.08	4286.88	4572.67
3666.67	4000.00	4333.33	4666.67	5000.00	5333.33
244.63	4630.50	5016.38	5402.25	5788.13	6174.00
880.33	5324.00	5767.67	6211.33	6655.00	7098.67
576.54	6083.50	6590.46	7097.42	7604.38	8111.33
336.00	6912.00	7488.00	8064.00	8640.00	9216.00
161.46	7812.50	8463.54	9114.58	9765.63	10416.67
055.67	8788.00	9520.33	10252.67	10985.00	11717.33
021.38	9841.50	10661.63	11481.75	12301.88	13122.00
061.33	10976.00	11890.67	12805.33	13720.00	14634.67
178.29	12194.50	13210.71	14226.92	15243.12	16250.33
375.00	13500.00	14625.00	15750.00	16875.00	18000.00

ble may be used in computing *the moments of inertia of plate girders,* other *compound sections in which plates are used. See pages* 341 and

Table II.* Areas and Radii of Gyration of Hollow-Round Sec



$$\text{Area} = \frac{\pi (D^2 - d^2)}{4} = 0.7854\,(D^2 - d^2) \text{ sq in}$$

$$\text{Radius of gyration} = \frac{\sqrt{D^2 + d^2}}{4} \text{ in}$$

Diam. D, inches	A and r	Thickness t in inches						
		¼	5/16	⅜	½	⅝	¾	⅞
2	A	1.37	1.66	......	......	......	......	......
	r	0.63	0.61	......	......	......	......	......
3	A	2.16	2.64	......	......	......	......	......
	r	0.98	0.96	......	......	......	......	......
4	A	2.95	3.62	4.27	5.50	......	......	......
	r	1.33	1.31	1.29	1.25	......	......	......
5	A	3.73	4.60	5.45	7.07	8.59	10.01	......
	r	1.68	1.66	1.64	1.60	1.56	1.53	......
6	A	4.52	5.58	6.63	8.64	10.55	12.37	14.09
	r	2.03	2.01	1.99	1.95	1.91	1.88	1.84
7	A	5.30	6.57	7.80	10.21	12.52	14.73	16.84
	r	2.39	2.37	2.35	2.30	2.27	2.23	2.19
8	A	6.09	7.55	8.98	11.78	14.48	17.08	19.59
	r	2.74	2.72	2.70	2.66	2.62	2.58	2.54
9	A	6.87	8.53	10.16	13.35	16.44	19.44	22.33
	r	3.09	3.07	3.05	3.01	2.97	2.93	2.89
10	A	7.66	9.51	11.34	14.92	18.41	21.79	25.08
	r	3.45	3.43	3.41	3.36	3.32	3.28	3.24
11	A	8.44	10.49	12.52	16.49	20.37	24.15	27.83
	r	3.80	3.78	3.76	3.72	3.67	3.63	3.59
12	A	9.23	11.47	13.70	18.06	22.33	26.51	30.58
	r	4.16	4.13	4.11	4.07	4.03	3.99	3.95
13	A	10.01	12.46	14.87	19.63	24.30	28.86	33.33
	r	4.51	4.49	4.47	4.42	4.38	4.34	4.30
14	A	10.80	13.44	16.05	21.21	26.26	31.22	36.08
	r	4.86	4.84	4.82	4.78	4.73	4.69	4.65
15	A	11.58	14.42	17.23	22.78	28.23	33.58	38.83
	r	5.22	5.19	5.17	5.13	5.09	5.05	5.00
16	A	12.37	15.40	18.41	24.35	30.19	35.93	41.58
	r	5.57	5.55	5.53	5.48	5.44	5.40	5.36
17	A	13.16	16.38	19.59	25.92	32.15	38.29	44.33
	r	5.92	5.90	5.88	5.84	5.79	5.75	5.71
18	A	13.94	17.36	20.76	27.49	34.12	40.64	47.07
	r	6.28	6.25	6.23	6.19	6.15	6.10	6.06
19	A	14.73	18.35	21.94	29.06	36.08	43.00	49.82
	r	6.63	6.61	6.59	6.54	6.50	6.46	6.42
20	A	15.51	19.33	23.12	30.63	38.04	45.36	52.57
	r	6.98	6.96	6.94	6.90	6.85	6.81	6.77

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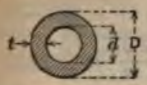
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitt

e II* (Continued). Areas and Radii of Gyration of Hollow-Round Sections



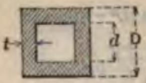
$$\text{Area} = \frac{\pi (D^2 - d^2)}{4} = 0.7854\,(D^2 - d^2) \text{ sq in}$$

$$\text{Radius of gyration} = \frac{\sqrt{D^2 + d^2}}{4} \text{ in}$$

am. hes	*A* and *r*	Thickness *t* in inches							
		1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2
	A	......	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......	......
	A	......	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......	......
	A	......	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......	......
	A	......	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......	......
	A	......	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......	......
7	*A*	20.76	22.58	......	......	......	......	......	......
	r	2.12	2.08	......	......	......	......	......	......
8	*A*	24.30	26.51	28.62	30.63	......	......	......	......
	r	2.46	2.43	2.39	2.36	......	......	......	......
9	*A*	27.83	30.43	32.94	35.34	37.65	39.86	......	......
	r	2.81	2.78	2.74	2.70	2.67	2.64	......	......
10	*A*	31.37	34.36	37.26	40.06	42.76	45.36	47.86	50.27
	r	3.16	3.13	3.09	3.05	3.02	2.98	2.95	2.92
11	*A*	34.90	38.29	41.58	44.77	47.86	50.85	53.75	56.55
	r	3.51	3.48	3.44	3.40	3.36	3.33	3.29	3.26
12	*A*	38.44	41.22	45.90	49.48	52.97	56.35	59.64	62.83
	r	3.87	3.83	3.79	3.75	3.71	3.68	3.64	3.61
13	*A*	41.97	46.14	50.22	54.19	58.07	61.85	65.53	69.12
	r	4.22	4.18	4.14	4.10	4.06	4.03	5.99	3.95
14	*A*	45.50	50.07	54.54	58.91	63.18	67.35	71.42	75.40
	r	4.57	4.53	4.49	4.45	4.41	4.38	4.34	4.30
15	*A*	49.04	54.00	58.86	63.62	68.28	72.85	77.31	81.68
	r	4.92	4.88	4.84	4.80	4.76	4.73	4.69	4.65
16	*A*	52.57	57.92	63.18	68.33	73.39	78.34	83.20	87.97
	r	5.27	5.23	5.19	5.15	5.11	5.08	5.04	5.00
17	*A*	56.11	61.85	67.50	73.04	78.49	83.84	89.09	94.25
	r	5.63	5.59	5.55	5.51	5.47	5.43	5.39	5.35
18	*A*	59.61	65.78	71.82	77.75	83.60	89.34	94.98	100.53
	r	5.98	5.94	5.90	5.86	5.82	5.78	5.74	5.70
19	*A*	63.18	69.70	76.13	82.47	88.70	94.84	100.87	106.82
	r	6.33	6.29	6.25	6.21	6.17	6.13	6.09	6.05
20	*A*	66.71	73.63	80.45	87.18	93.81	100.33	106.77	113.10
	r	6.69	6.64	6.60	6.56	6.52	6.48	6.44	6.40

* *From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table III.* **Areas and Radii of Gyration of Hollow-Square Sectio**



Area = $(D^2 - d^2)$ sq in

Radius of gyration = $\sqrt{\frac{D^2 + d^2}{12}}$ in

Side D, inches	A and r	Thickness t in inches						
		1/4	5/16	3/8	1/2	5/8	3/4	7/8
2	A	1.75	2.11	......	......	......	......	......
	r	0.72	0.70	......	......	......	......	......
3	A	2.75	3.36	......	......	......	......	......
	r	1.13	1.10	......	......	......	......	......
4	A	3.75	4.61	5.44	7.00	......	......	......
	r	1.53	1.51	1.49	1.44	......	......	......
5	A	4.75	5.86	6.94	9.00	10.94	12.75	......
	r	1.94	1.92	1.89	1.85	1.80	1.76	......
6	A	5.75	7.11	8.44	11.00	13.44	15.75	17.94
	r	2.35	2.33	2.30	2.25	2.21	2.17	2.12
7	A	6.75	8.36	9.94	13.00	15.94	18.75	21.44
	r	2.76	2.73	2.71	2.66	2.62	2.57	2.53
8	A	7.75	9.61	11.44	15.00	18.44	21.75	24.94
	r	3.17	3.14	3.12	3.07	3.02	2.98	2.93
9	A	8.75	10.86	12.94	17.00	20.94	24.75	28.44
	r	3.57	3.55	3.53	3.48	3.43	3.38	3.34
10	A	9.75	12.11	14.44	19.00	23.44	27.75	31.94
	r	3.98	3.96	3.93	3.88	3.84	3.79	3.74
11	A	10.75	13.36	15.94	21.00	25.94	30.75	35.44
	r	4.39	4.37	4.34	4.29	4.24	4.20	4.15
12	A	11.75	14.61	17.44	23.00	28.44	33.75	38.94
	r	4.80	4.77	4.75	4.70	4.65	4.60	4.56
13	A	12.75	15.86	18.94	25.00	30.94	36.75	42.44
	r	5.21	5.18	5.16	5.11	5.06	5.01	4.96
14	A	13.75	17.11	20.44	27.00	33.44	39.75	45.94
	r	5.61	5.59	5.56	5.51	5.47	5.42	5.37
15	A	14.75	18.36	21.94	29.00	35.94	42.75	49.44
	r	6.02	6.00	5.97	5.92	5.87	5.83	5.78
16	A	15.75	19.61	23.44	31.00	38.44	45.75	52.94
	r	6.43	6.41	6.38	6.33	6.28	6.23	6.19
17	A	16.75	20.86	24.94	33.00	40.94	48.75	56.44
	r	6.84	6.81	6.79	6.74	6.69	6.64	6.59
18	A	17.75	22.11	26.44	35.00	43.44	51.75	59.94
	r	7.25	7.22	7.20	7.15	7.10	7.05	7.00
19	A	18.75	23.36	27.94	37.00	45.94	54.75	63.44
	r	7.66	7.63	7.61	7.56	7.51	7.46	7.41
20	A	19.75	24.61	29.44	39.00	48.44	57.75	66.94
	r	8.06	8.04	8.01	7.96	7.91	7.87	7.82

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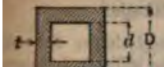
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

III * (Continued). Areas and Radii of Gyration of Hollow-Square Sections



Area = $(D^2 - d^2)$ sq in

Radius of gyration = $\sqrt{\frac{D^2 + d^2}{12}}$ in

A and *r*	Thickness *t* in inches							
	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2
A	......	......	......	......	......	......	......	......
r	......	......	......	......	......	......	......	......
A	......	......	......	......	......	......	......	......
r	......	......	......	......	......	......	......	......
A	......	......	......	......	......	......	......	......
r	......	......	......	......	......	......	......	......
A	......	......	......	......	......	......	......	......
r	......	......	......	......	......	......	......	......
A	......	......	......	......	......	......	......	......
r	......	......	......	......	......	......	......	......
A	26.44	28.75	......	......	......	......	......	......
r	2.44	2.40	......	......	......	......	......	......
A	30.94	33.75	36.44	39.00	......	......	......	......
r	2.84	2.80	2.76	2.72	......	......	......	......
A	35.44	38.75	41.94	45.00	47.94	50.75	......	......
r	3.25	3.20	3.16	3.12	3.08	3.05	......	......
A	39.94	43.75	47.44	51.00	54.44	57.75	60.94	64.00
r	3.65	3.61	3.57	3.52	3.48	3.44	3.40	3.37
A	44.44	48.75	52.94	57.00	60.94	64.75	68.44	72.00
r	4.06	4.01	3.97	3.93	3.88	3.84	3.80	3.76
A	48.94	53.75	58.44	63.00	67.44	71.75	75.94	80.00
r	4.46	4.42	4.37	4.33	4.29	4.25	4.20	4.16
A	53.44	58.75	63.94	69.00	73.94	78.75	83.44	88.00
r	4.87	4.82	4.78	4.74	4.69	4.65	4.61	4.56
A	57.94	63.75	69.44	75.00	80.44	85.75	90.94	96.00
r	5.28	5.23	5.18	5.14	5.10	5.05	5.01	4.97
A	62.44	68.75	74.94	81.00	86.94	92.75	98.44	104.00
r	5.68	5.64	5.59	5.55	5.50	5.46	5.41	5.37
A	66.94	73.75	80.44	87.00	93.44	99.75	105.94	112.00
r	6.09	6.04	6.00	5.95	5.91	5.86	5.82	5.77
A	71.44	78.75	85.94	93.00	99.94	106.75	113.44	120.00
r	6.50	6.45	6.40	6.36	6.31	6.27	6.23	6.18
A	75.94	83.75	91.44	99.00	106.44	113.75	120.94	128.00
r	6.90	6.86	6.81	6.76	6.72	6.67	6.63	6.58
A	80.44	88.75	96.94	105.00	112.94	120.75	128.44	136.00
r	7.31	7.26	7.22	7.17	7.12	7.08	7.03	6.99
A	84.94	93.75	102.44	111.00	119.44	127.75	135.94	144.00
r	7.72	7.67	7.62	7.58	7.53	7.49	7.44	7.39

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

6. Dimensions, Moments of Inertia, Radii of Gyration and Se Moduli of Standard Structural Shapes

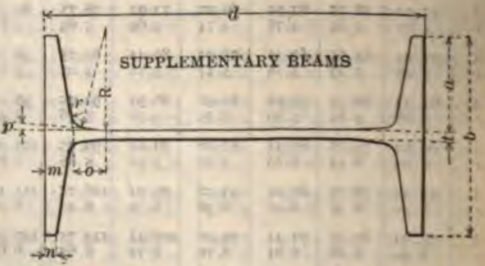
Explanation of Tables. As in using structural-steel shapes the practically confined to such shapes as are rolled by the mills, it is have at hand the dimensions and properties of those shapes in order late the necessary sizes to meet special requirements for strength and conditions of economy and framing. Since 1890 great changes have been both in the materials and in the shapes of the standard sections. The mills which manufacture the most complete assortment of structural those of the Carnegie Steel Company, the Cambria Steel Company, the & Laughlin Steel Company and the Bethlehem Steel Company. In products of these mills, especially beams and channels, are respectively in shape. This is particularly true of the shapes rolled by the first three companies named.

The standard steel beams and channels considered in the following rolled by all of these mills, with the exception of those of the Bethlehem Company. With a few exceptions the following tables of properties ard structural shapes are adapted from the 1915 edition of the Pocket panion of the Carnegie Steel Company. It may be well to state that the of properties for the various structural shapes, published by the companies above, do not agree exactly, even for the same weights, but the differen not of practical importance. The tables of the Cambria Steel Compa of the Carnegie Steel Company for beams and channels agree more clos angles are very extensively used for a great many purposes, the propert given for all sizes rolled and also a table showing from which mills the sizes may be obtained. Naturally it will generally be advantageous to size that is rolled by several mills.

Tables XV, XVI and XVII will be found very convenient when co the strength of struts formed of pairs of angles, and Table XVIII wh puting the same for pairs of channels.

Standard Steel Beams and Channels*

COMMON DIMENSIONS



$b = d/6 + 3$ $o = 3.82\,t - 0.10$

$t = 0.01125\,d + 0.12$ p, computed

$n = 0.01875\,d + 0.09$ $r = 1.48\,t + 0.02$

$m = n + (b - t)/12$ $R = 16.78\,t - 0.66$

Slope of flange, 1 : 6 = $16\frac{2}{3}\%$ = 9° 27′ 42″

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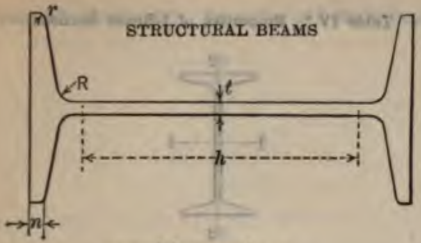
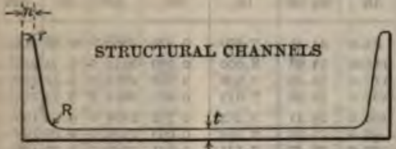
* From Pocket Companion, 1915 Edition, Carnegie Steel Company,



n = minimum web = t
R = minimum web + 0.10
$r = \frac{9}{10}$ minimum web
h = distance between flange-fillets
Slope of flange, $1:6 = 16\frac{2}{3}\% = 9^\circ\, 27'\, 42''$

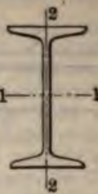


n = minimum web = t
R = minimum web + 0.10
$r = \frac{9}{10}$ minimum web
Slope of flange, $1:6 = 16\frac{2}{3}\% = 9^\circ\, 27'\, 42''$

nensions are in inches and apply only to the minimum-weight beams or

sions given for structural beams are those adopted in 1896, by the
on of American Steel Manufacturers and apply to all beam-sections
the pages of the Carnegie Steel Company's Pocket Companion, 1915
except the American Standard beam-sections B 1, B 2 and B 3, beam-
B 24 and B 81, and supplementary beams B 31 to B 38, inclusive.
sions shown for structural channels are those adopted by the Associ-
American Steel Manufacturers and apply to all structural channel-

Table IV.[*] Properties of I-Beam Sections



Section index	Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			
						I	r	I/c	I
	in	lb	sq in	in	in	in^4	in	in^3	in^4
B 31	27	83.0	24.41	7.500	0.424	2888.6	10.88	214.0	53.
B 24	24	115.0	33.98	8.000	0.750	2955.5	9.33	246.3	83.
		110.0	32.48	7.938	0.688	2883.5	9.42	240.3	81.
		105.0	30.98	7.875	0.625	2811.5	9.53	234.3	78.
B 1	24	100.0	29.41	7.254	0.754	2379.6	9.00	198.3	48.
		95.0	27.94	7.193	0.693	2309.0	9.09	192.4	47.
		90.0	26.47	7.131	0.631	2238.4	9.20	186.5	45.
		85.0	25.00	7.070	0.570	2167.8	9.31	180.7	44.
		80.0	23.32	7.000	0.500	2087.2	9.46	173.9	42.
B 32	24	69.5	20.44	7.000	0.390	1928.0	9.71	160.7	39.
B 33	21	57.5	16.85	6.500	0.357	1227.5	8.54	116.9	28.
B 2	20	100.0	29.41	7.284	0.884	1655.6	7.50	165.6	52.
		95.0	27.94	7.210	0.810	1606.6	7.58	160.7	50.
		90.0	26.47	7.137	0.737	1557.6	7.67	155.8	49.
		85.0	25.00	7.063	0.663	1508.5	7.77	150.9	47.
		80.0	23.73	7.000	0.600	1466.3	7.86	146.6	45.
B 3	20	75.0	22.06	6.399	0.649	1268.8	7.58	126.9	30.
		70.0	20.59	6.325	0.575	1219.8	7.70	122.0	29.
		65.0	19.08	6.250	0.500	1169.5	7.83	117.0	27.
B 81	18	90.0	26.47	7.245	0.807	1260.4	6.90	140.0	52.
		85.0	25.00	7.163	0.725	1220.7	6.99	135.6	50.
		80.0	23.53	7.082	0.644	1181.0	7.09	131.2	48.
		75.0	22.05	7.000	0.562	1141.3	7.19	126.8	46.
B 80	18	70.0	20.59	6.259	0.719	921.2	6.69	102.4	24.
		65.0	19.12	6.177	0.637	881.5	6.79	97.9	23.
		60.0	17.65	6.095	0.555	841.8	6.91	93.5	22.
		55.0	15.93	6.000	0.460	795.6	7.07	88.4	21.
B 34	18	46.0	13.53	6.000	0.322	733.2	7.36	81.5	19.
B 5	15	75.0	22.06	6.292	0.882	691.2	5.60	92.2	30.
		70.0	20.59	6.194	0.784	663.7	5.68	88.5	29.
		65.0	19.12	6.096	0.686	636.1	5.77	84.8	27.
		60.0	17.67	6.000	0.590	609.0	5.87	81.2	26.
B 7	15	55.0	16.18	5.746	0.656	511.0	5.62	68.1	17.
		50.0	14.71	5.648	0.558	483.4	5.73	64.5	16.
		45.0	13.24	5.550	0.460	455.9	5.87	60.8	15.
		42.0	12.48	5.500	0.410	441.8	5.95	58.9	14.
B 35	15	36.0	10.63	5.500	0.289	405.1	6.17	54.0	13.

NOTE. The exponential figures used with *I* and *I/c* denote the
" dimensions " of these quantities, that is, the number of times the
pears as a factor in the quantities.

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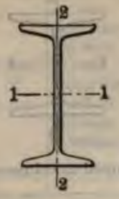
[*] From Pocket Companion, 1915 Edition, Carnegie Steel Company

Table IV* (Continued). Properties of I-Beam Section

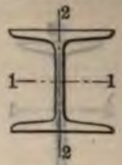


Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			Axis 2-2		
					I	*r*	*I/c*	*I*	*r*	*I/c*
in	lb	sq in	in	in	in⁴	in	in³	in⁴	in	in³
12	55.0	16.18	5.611	0.821	321.0	4.45	53.5	17.5	1.04	6.2
	50.0	14.71	5.489	0.699	303.4	4.54	50.6	16.1	1.05	5.9
	45.0	13.24	5.366	0.576	285.7	4.65	47.6	14.9	1.06	5.6
	40.0	11.84	5.250	0.460	269.0	4.77	44.8	13.8	1.08	5.3
12	35.0	10.29	5.086	0.436	228.3	4.71	38.0	10.1	0.99	4.0
	31.5	9.26	5.000	0.350	215.8	4.83	36.0	9.5	1.01	3.8
12	27.5	8.04	5.000	0.255	199.6	4.98	33.3	8.7	1.04	3.5
10	40.0	11.76	5.099	0.749	158.7	3.67	31.7	9.5	0.90	3.7
	35.0	10.29	4.952	0.602	146.4	3.77	29.3	8.5	0.91	3.4
	30.0	8.82	4.805	0.455	134.2	3.90	26.8	7.7	0.93	3.2
	25.0	7.37	4.660	0.310	122.1	4.07	24.4	6.9	0.97	3.0
10	22.0	6.52	4.670	0.232	113.9	4.18	22.8	6.4	0.99	2.7
9	35.0	10.29	4.772	0.732	111.8	3.29	24.8	7.3	0.84	3.1
	30.0	8.82	4.609	0.569	101.9	3.40	22.6	6.4	0.85	2.8
	25.0	7.35	4.446	0.406	91.9	3.54	20.4	5.7	0.88	2.5
	21.0	6.31	4.330	0.290	84.9	3.67	18.9	5.2	0.90	2.4
8	25.5	7.50	4.271	0.541	68.4	3.02	17.1	4.8	0.80	2.2
	23.0	6.76	4.179	0.449	64.5	3.09	16.1	4.4	0.81	2.1
	20.5	6.03	4.087	0.357	60.6	3.17	15.2	4.1	0.82	2.0
	18.0	5.33	4.000	0.270	56.9	3.27	14.2	3.8	0.84	1.9
8	17.5	5.15	4.330	0.210	58.3	3.37	14.6	4.5	0.93	2.1
7	20.0	5.88	3.868	0.458	42.2	2.68	12.1	3.2	0.74	1.7
	17.5	5.15	3.763	0.353	39.2	2.76	11.2	2.9	0.76	1.6
	15.0	4.42	3.660	0.250	36.2	2.86	10.4	2.7	0.78	1.5
6	17.25	5.07	3.575	0.475	26.2	2.27	8.7	2.4	0.68	1.3
	14.75	4.34	3.452	0.352	24.0	2.35	8.0	2.1	0.69	1.2
	12.25	3.61	3.330	0.230	21.8	2.46	7.3	1.9	0.72	1.1
5	14.75	4.34	3.294	0.504	15.2	1.87	6.1	1.7	0.63	1.0
	12.25	3.60	3.147	0.357	13.6	1.94	5.5	1.5	0.63	0.92
	9.75	2.87	3.000	0.210	12.1	2.05	4.8	1.2	0.65	0.82
4	10.5	3.09	2.880	0.410	7.1	1.52	3.6	1.0	0.57	0.70
	9.5	2.79	2.807	0.337	6.8	1.55	3.4	0.93	0.58	0.66
	8.5	2.50	2.733	0.263	6.4	1.59	3.2	0.85	0.58	0.62
	7.5	2.21	2.660	0.190	6.0	1.64	3.0	0.77	0.59	0.58
3	7.5	2.21	2.521	0.361	2.9	1.15	1.9	0.60	0.52	0.48
	6.5	1.91	2.423	0.263	2.7	1.19	1.8	0.53	0.52	0.44
	5.5	1.63	2.330	0.170	2.5	1.23	1.7	0.46	0.53	0.40

"Note" with table on *preceding page.*

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table V.* Properties of H-Beam Sections



These may be employed as columns, using the axis 2-2

Section-index	Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			A
						I	*r*	*I/c*	*I*
	in	lb	sq in	in	in	in^4	in	in^3	in^4
H 4	8	34.0	10.00	8.0	0.375	115.4	3.40	28.9	35.1
H 3	6	23.8	7.00	6.0	0.313	45.1	2.54	15.0	14.7
H 2	5	18.7	5.50	5.0	0.313	23.8	2.08	9.5	7.9
H 1	4	13.6	4.00	4.0	0.313	10.7	1.63	5.3	3.6

See " Note " with Table IV, page 354.

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitt

Table VI.* Properties of Bethlehem I-Beam Sections

pth f am	Weight per foot	Area of section	Thickness of web	Width of flange	Increase of web and flange for each lb increase of weight	Neutral axis perpendicular to web at center			Neutral axis coincident with center line of web	
						I	r	I/c	I	r
n	lb	sq in	in	in	in	in^4	in	in^3	in^4	in
30	120.0	35.30	0.540	10.500	0.010	5239.6	12.18	349.3	165.0	2.16
28	105.0	30.88	0.500	10.000	0.011	4014.1	11.40	286.7	131.5	2.06
26	90.0	26.49	0.460	9.500	0.011	2977.2	10.60	229.0	101.2	1.95
24	84.0	24.80	0.460	9.250	0.012	2381.9	9.80	198.5	91.1	1.92
24	83.0	24.59	0.520	9.130	0.012	2240.9	9.55	186.7	78.0	1.78
24	73.0	21.47	0.390	9.000	0.012	2091.0	9.87	174.3	74.4	1.86
20	82.0	24.17	0.570	8.890	0.015	1559.8	8.03	156.0	79.9	1.82
20	72.0	21.37	0.430	8.750	0.015	1466.5	8.28	146.7	75.9	1.88
20	69.0	20.26	0.520	8.145	0.015	1268.9	7.91	126.9	51.2	1.59
20	64.0	18.86	0.450	8.075	0.015	1222.1	8.05	122.2	49.8	1.62
20	59.0	17.36	0.375	8.000	0.015	1172.2	8.22	117.2	48.3	1.66
18	59.0	17.40	0.495	7.675	0.016	883.3	7.12	98.1	39.1	1.50
18	54.0	15.87	0.410	7.590	0.016	842.0	7.28	93.6	37.7	1.54
18	52.0	15.24	0.375	7.555	0.016	825.0	7.36	91.7	37.1	1.56
18	48.5	14.25	0.320	7.500	0.016	798.3	7.48	88.7	36.2	1.59
15	71.0	20.95	0.520	7.500	0.020	796.2	6.16	106.2	61.3	1.71
15	64.0	18.81	0.605	7.195	0.020	664.9	5.95	88.6	41.9	1.49
15	54.0	15.88	0.410	7.000	0.020	610.0	6.20	81.3	38.3	1.55
15	46.0	13.52	0.440	6.810	0.020	484.8	5.99	64.6	25.2	1.36
15	41.0	12.02	0.340	6.710	0.020	456.7	6.16	60.9	24.0	1.41
15	38.0	11.27	0.290	6.660	0.020	442.6	6.27	59.0	23.4	1.44
12	36.0	10.61	0.310	6.300	0.025	269.2	5.04	44.9	21.3	1.42
12	32.0	9.44	0.335	6.205	0.025	228.5	4.92	38.1	16.0	1.30
12	28.5	8.42	0.250	6.120	0.025	216.2	5.07	36.0	15.3	1.35
10	28.5	8.34	0.390	5.990	0.029	134.6	4.02	26.9	12.1	1.21
10	23.5	6.94	0.250	5.850	0.029	122.9	4.21	24.6	11.2	1.27
9	24.0	7.04	0.365	5.555	0.033	92.1	3.62	20.5	8.8	1.12
9	20.0	6.01	0.250	5.440	0.033	85.1	3.76	18.9	8.2	1.17
8	19.5	5.78	0.325	5.325	0.037	60.6	3.24	15.1	6.7	1.08
8	17.5	5.18	0.250	5.250	0.037	57.4	3.33	14.3	6.4	1.11

See "Note" with Table IV, page 354.

* Adapted from Catalogue of Structural *Shapes, 1909 Edition, Bethlehem Steel* mpany, Bethlehem, *Pa.*

Table VII.* Properties of Bethlehem Girder-Beam Sections

Depth of beam	Weight per foot	Area of section	Thickness of web	Width of flange	Increase of web and flange for each pound increase of weight	Neutral axis perpendicular to web at center		
						I	r	I/c
in	lb	sq in	in	in	in	in^4	in	in^3
30	200.0	58.71	0.750	15.00	0.010	9150.6	12.48	610.0
30	180.0	53.00	0.690	13.00	0.010	8194.5	12.43	546.3
28	180.0	52.86	0.690	14.35	0.011	7264.7	11.72	518.9
28	165.0	48.47	0.660	12.50	0.011	6562.7	11.64	468.8
26	160.0	46.91	0.630	13.60	0.011	5620.8	10.95	432.4
26	150.0	43.94	0.630	12.00	0.011	5153.9	10.83	396.5
24	140.0	41.16	0.600	13.00	0.012	4201.4	10.10	350.1
24	120.0	35.38	0.530	12.00	0.012	3607.3	10.10	300.6
20	140.0	41.19	0.640	12.50	0.015	2934.7	8.44	293.5
20	112.0	32.81	0.550	12.00	0.015	2342.1	8.45	234.2
18	92.0	27.12	0.480	11.50	0.016	1591.4	7.66	176.8
15	140.0	41.27	0.800	11.75	0.020	1592.7	6.21	212.4
15	104.0	30.50	0.600	11.25	0.020	1220.1	6.32	162.7
15	73.0	21.49	0.430	10.50	0.020	883.4	6.41	117.8
12	70.0	20.58	0.460	10.00	0.025	538.8	5.12	89.8
12	55.0	16.18	0.370	9.75	0.025	432.0	5.17	72.0
10	44.0	12.95	0.310	9.00	0.030	244.2	4.34	48.8
9	38.0	11.22	0.300	8.50	0.033	170.9	3.90	38.0
8	32.5	9.54	0.290	8.00	0.037	114.4	3.46	28.6

See " Note " with Table IV, page 354.

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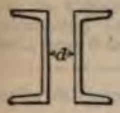
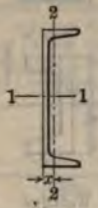
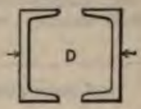
* Adapted from Catalogue of Structural Shapes, 1909 Edition, Bethl
Company, Bethlehem, Pa.

Table VIII.* Properties of Channel-Sections





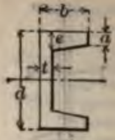


Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			Axis 2-2			x	D	d
				I	r	I/c	I	r	I/c			
lb	sq in	in	in	in⁴	in	in³	in⁴	in	in³	in		
55.0	16.18	3.818	0.818	430.2	5.16	57.4	12.2	0.87	4.1	0.82	12.82	9.50
50.0	14.71	3.720	0.720	402.7	5.23	53.7	11.2	0.87	3.8	0.80	12.73	9.43
45.0	13.24	3.622	0.622	375.1	5.32	50.0	10.3	0.88	3.6	0.79	12.42	9.15
40.0	11.76	3.524	0.524	347.5	5.43	46.3	9.4	0.89	3.4	0.78	12.23	8.92
35.0	10.29	3.426	0.426	319.9	5.58	42.7	8.5	0.91	3.2	0.79	12.06	8.71
33.0	9.90	3.400	0.400	312.6	5.62	41.7	8.2	0.91	3.2	0.79	11.96	8.53
40.0	11.76	3.418	0.758	196.9	4.09	32.8	6.6	0.75	2.5	0.72	10.63	7.67
35.0	10.29	3.296	0.636	179.3	4.17	29.9	5.9	0.76	2.3	0.69	10.21	7.36
30.0	8.82	3.173	0.513	161.7	4.28	26.9	5.2	0.77	2.1	0.68	9.91	7.07
25.0	7.35	3.050	0.390	144.0	4.43	24.0	4.5	0.79	1.9	0.68	9.73	6.81
20.5	6.03	2.940	0.280	128.1	4.61	21.4	3.9	0.81	1.7	0.70	9.62	6.60
35.0	10.29	3.183	0.823	115.5	3.35	23.1	4.7	0.67	1.9	0.70	9.02	6.33
30.0	8.82	3.036	0.676	103.2	3.42	20.7	4.0	0.67	1.7	0.65	8.54	5.97
25.0	7.35	2.889	0.529	91.0	3.52	18.2	3.4	0.68	1.5	0.62	8.28	5.67
20.0	5.88	2.742	0.382	78.7	3.66	15.7	2.9	0.70	1.3	0.61	8.14	5.40
15.0	4.46	2.600	0.240	66.9	3.87	13.4	2.3	0.72	1.2	0.64	8.09	5.17
25.0	7.35	2.815	0.615	70.7	3.10	15.7	3.0	0.64	1.4	0.62	8.19	5.63
20.0	5.88	2.652	0.452	60.8	3.21	13.5	2.5	0.65	1.2	0.59	7.98	5.49
15.0	4.41	2.488	0.288	50.9	3.40	11.3	2.0	0.67	1.0	0.59	7.59	5.12
13.25	3.89	2.430	0.230	47.3	3.49	10.5	1.8	0.67	0.97	0.61	7.43	4.84
21.25	6.25	2.622	0.582	47.8	2.77	11.9	2.3	0.60	1.1	0.59	7.37	4.94
18.75	5.51	2.530	0.490	43.8	2.82	11.0	2.0	0.60	1.0	0.57	7.07	4.72
16.25	4.78	2.439	0.399	39.9	2.89	10.0	1.8	0.61	0.95	0.56	6.89	4.54
13.75	4.04	2.347	0.307	36.0	2.98	9.0	1.6	0.62	0.87	0.56	6.77	4.38
11.25	3.35	2.260	0.220	32.3	3.11	8.1	1.3	0.63	0.79	0.58	6.71	4.23
19.75	5.81	2.513	0.633	33.2	2.39	9.5	1.9	0.56	0.96	0.58	6.53	4.22
17.25	5.07	2.408	0.528	30.2	2.44	8.6	1.6	0.57	0.87	0.56	6.21	3.99
14.75	4.34	2.303	0.423	27.2	2.50	7.8	1.4	0.57	0.79	0.54	6.07	3.80
12.25	3.60	2.198	0.318	24.2	2.59	6.9	1.2	0.58	0.71	0.53	5.99	3.64
9.75	2.85	2.090	0.210	21.1	2.72	6.0	0.98	0.59	0.63	0.55	5.94	3.48
15.5	4.56	2.283	0.563	19.5	2.07	6.5	1.3	0.53	0.74	0.55	5.71	3.52
13.0	3.82	2.160	0.440	17.3	2.13	5.8	1.1	0.53	0.65	0.52	5.42	3.28
10.5	3.09	2.038	0.318	15.1	2.21	5.0	0.88	0.53	0.57	0.50	5.29	3.09
8.0	2.38	1.920	0.200	13.0	2.34	4.3	0.70	0.54	0.50	0.52	5.23	2.91
11.5	3.38	2.037	0.477	10.4	1.75	4.2	0.82	0.49	0.54	0.51	4.87	2.79
9.0	2.65	1.890	0.330	8.9	1.83	3.6	0.64	0.49	0.45	0.48	4.62	2.56
6.5	1.95	1.750	0.190	7.4	1.95	3.0	0.48	0.50	0.38	0.49	4.51	2.34
7.25	2.13	1.725	0.325	4.6	1.46	2.3	0.44	0.46	0.35	0.46	4.04	2.06
6.25	1.84	1.652	0.252	4.2	1.51	2.1	0.38	0.45	0.32	0.46	3.93	1.96
5.25	1.55	1.580	0.180	3.8	1.56	1.9	0.32	0.45	0.29	0.46	3.84	1.85
6.0	1.76	1.602	0.362	2.1	1.08	1.4	0.31	0.42	0.27	0.46	3.22	1.31
5.0	1.47	1.504	0.264	1.8	1.12	1.2	0.25	0.42	0.24	0.44	3.12	1.19
4.0	1.19	1.410	0.170	1.6	1.17	1.1	0.20	0.41	0.21	0.44	3.07	1.07

"Note" with *Table IV, page 354.*

earranged from Pocket Companion, 1915 Edition, Carnegie Steel Company,
rgh, Pa.

Table IX.* **Dimensions of Sections and Weights of Small Grooved**
Channels



Section-index	Size of section, in	Width of flange, in	Thickness of web, in	Weight foot, lb
C-164	2⅛	13/16	¼	2.53
C-165	2⅛	¾	3/16	2.09
C-166	2⅛	11/16	⅛	1.63
C-183	2	⅝	¼	2.11
C-184	2	9/16	3/16	1.68
C-185	2	½	⅛	1.26
C-190	1¾	11/16	3/16	1.71
C-191	1¾	⅝	⅛	1.33
C-193	1¾	17/32	5/32	1.33
C-195	1¾	⅜	⅛	0.96
C-197	1½	⅝	3/16	1.47
C-199	1¼	½	⅛	0.93
C-200	1⅛	9/16	3/16	1.12
C-203	1	½	⅛	0.83
C-207	1	25/64	⅛	0.71
C-213	⅞	7/16	⅛	0.66
C-217	⅞	⅜	⅛	0.58
C-219	¾	⅜	⅛	0.54
C-221	¾	11/32	3/32	0.40
C-223	⅝	5/16	⅛	0.43

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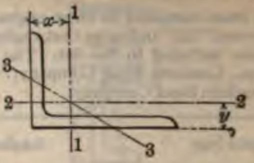
* Rolled by the Jones & Laughlin Steel Company, Pittsburgh, Pa.

Table X. Sizes and Makes of Rolled Steel Angles

ing table has been compiled to show angles of various sizes rolled by
panies. The word "all" indicates that angles of the sizes mentioned are
the companies included in the list. The abbreviations refer to the
mpanies: Cam., Cambria Steel Company; Car., Carnegie Steel Com-
., Jones & Laughlin Steel Company.

les with unequal legs		Angles with equal legs	
ches	Companies	Sizes in inches	Companies
	Cam. and Car.	8 ×8	All
	Car.	6 ×6	All
	All	5 ×5	All
	All	4½×4½	Cam.
	All	4 ×4	All
	All	3½×3½	All
	All	3¼×3¼	J. & L.
	All	3 ×3	All
	Car. and J. & L.	2¾×2¾	Cam. and J. & L.
	All	2½×2½	All
	All	2¼×2¼	Cam. and J. & L.
	All	2 ×2	All
	All	1¾×1¾	All
	J. & L.	1½×1½	All
	J. & L.	1⅜×1⅜	J. & L.
	All	1¼×1¼	All
	All	1³⁄₁₆×1³⁄₁₆	J. & L.
	All	1 ×1	All
	J. & L.	¾× ¾	J. & L.
	Car. and J. & L.		
	Cam.		
	J. & L.		
	Car. and J. & L.		
	Car. and J. & L.		
	J. & L.		
	Car.		
	J. & L.		
	J. & L.		
	Car.		
	J. & L.		
	Car.		
	J. & L.		
	J. & L.		
	J. & L.		
	J. & L.		

Table XL.* Properties of Angle-Sections. Unequal Legs

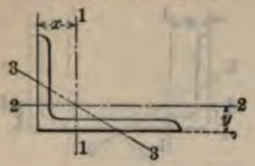


Size	Weight per foot	Area of section	Axis 1-1				Axis 2-2			
			I	r	I/c	x	I	r	I/c	y
in	lb	sq in	in^4	in	in^3	in	in^4	in	in^3	in
8×6 ×1	44.2	13.00	80.8	2.49	15.1	2.65	38.8	1.73	8.9	1.
8×6 ×15⁄16	41.7	12.25	76.6	2.50	14.3	2.63	36.8	1.73	8.4	1.
8×6 ×⅞	39.1	11.48	72.3	2.51	13.4	2.61	34.9	1.74	7.9	1.
8×6 ×13⁄16	36.5	10.72	67.9	2.52	12.5	2.59	32.8	1.75	7.4	1.
8×6 ×¾	33.8	9.94	63.4	2.53	11.7	2.56	30.7	1.76	6.9	1.
8×6 ×11⁄16	31.2	9.15	58.8	2.54	10.8	2.54	28.6	1.77	6.4	1.
8×6 ×⅝	28.5	8.36	54.1	2.54	9.9	2.52	26.3	1.77	5.9	1.
8×6 ×9⁄16	25.7	7.56	49.3	2.55	8.9	2.50	24.0	1.78	5.3	1.
8×6 ×½	23.0	6.75	44.3	2.56	8.0	2.47	21.7	1.79	4.8	1.
8×6 ×7⁄16	20.2	5.93	39.2	2.57	7.1	2.45	19.3	1.80	4.2	1.
8×3½×1	35.7	10.50	66.2	2.51	13.7	3.17	7.8	0.86	3.0	0.
8×3½×15⁄16	33.7	9.90	62.9	2.52	12.9	3.14	7.4	0.87	2.9	0.
8×3½×⅞	31.7	9.30	59.4	2.53	12.2	3.12	7.1	0.87	2.7	0.
8×3½×13⁄16	29.6	8.68	55.9	2.54	11.4	3.10	6.7	0.88	2.5	0.
8×3½×¾	27.5	8.06	52.3	2.55	10.6	3.07	6.3	0.88	2.3	0.
8×3½×11⁄16	25.3	7.43	48.5	2.56	9.8	3.05	5.9	0.89	2.2	0.
8×3½×⅝	23.2	6.80	44.7	2.57	9.0	3.03	5.4	0.90	2.0	0.
8×3½×9⁄16	21.0	6.15	40.8	2.57	8.2	3.00	5.0	0.90	1.8	0.
8×3½×½	18.7	5.50	36.7	2.58	7.3	2.98	4.5	0.91	1.6	0.
8×3½×7⁄16	16.5	4.84	32.5	2.59	6.4	2.95	4.1	0.92	1.5	0.
7×3½×1	32.3	9.50	45.4	2.19	10.6	2.71	7.5	0.89	3.0	0.
7×3½×15⁄16	30.5	8.97	43.1	2.19	10.0	2.69	7.2	0.89	2.8	0.
7×3½×⅞	28.7	8.42	40.8	2.20	9.4	2.66	6.8	0.90	2.6	0.
7×3½×13⁄16	26.8	7.87	38.4	2.21	8.8	2.64	6.5	0.91	2.5	0.
7×3½×¾	24.9	7.31	36.0	2.22	8.2	2.62	6.1	0.91	2.3	0.
7×3½×11⁄16	23.0	6.75	33.5	2.23	7.6	2.60	5.7	0.92	2.1	0.
7×3½×⅝	21.0	6.17	30.9	2.24	7.0	2.57	5.3	0.93	2.0	0.
7×3½×9⁄16	19.1	5.59	28.2	2.25	6.3	2.55	4.9	0.93	1.8	0.
7×3½×½	17.0	5.00	25.4	2.25	5.7	2.53	4.4	0.94	1.6	0.
7×3½×7⁄16	15.0	4.40	22.6	2.26	5.0	2.50	4.0	0.95	1.4	0.
7×3½×⅜	13.0	3.80	19.6	2.27	4.3	2.48	3.5	0.96	1.3	0.
6×4 ×1	30.6	9.00	30.8	1.85	8.0	2.17	10.8	1.09	3.8	1.
6×4 ×15⁄16	28.9	8.50	29.3	1.86	7.6	2.14	10.3	1.10	3.6	1.
6×4 ×⅞	27.2	7.98	27.7	1.86	7.2	2.12	9.8	1.11	3.4	1.
6×4 ×13⁄16	25.4	7.47	26.1	1.87	6.7	2.10	9.2	1.11	3.2	1.
6×4 ×¾	23.6	6.94	24.5	1.88	6.2	2.08	8.7	1.12	3.0	1.
6×4 ×11⁄16	21.8	6.40	22.8	1.89	5.8	2.06	8.1	1.13	2.8	1.
6×4 ×⅝	20.0	5.86	21.1	1.90	5.3	2.03	7.5	1.13	2.5	1.
6×4 ×9⁄16	18.1	5.31	19.3	1.90	4.8	2.01	6.9	1.14	2.3	1
6×4 ×½	16.2	4.75	17.4	1.91	4.3	1.99	6.3	1.15	2.1	0
6×4 ×7⁄16	14.3	4.18	15.5	1.92	3.8	1.96	5.6	1.16	1.8	0
6×4 ×⅜	12.3	3.61	13.5	1.93	3.3	1.94	4.9	1.17	1.6	0

See "Note" with Table IV, page 354.

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pi

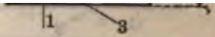
XI* (Continued). Properties of Angle-Sections. Unequal Legs



	Weight per foot	Area of section	Axis 1-1				Axis 2-2				Axis 3-3
			I	r	I/c	x	I	r	I/c	y	r_{min}
	lb	sq in	in^4	in	in^3	in	in^4	in	in^3	in	in
1	28.9	8.50	29.2	1.85	7.8	2.26	7.2	0.92	2.9	1.01	0.74
15/16	27.3	8.03	27.8	1.86	7.4	2.24	6.9	0.93	2.7	0.99	0.74
7/8	25.7	7.55	26.4	1.87	7.0	2.22	6.6	0.93	2.6	0.97	0.75
13/16	24.0	7.06	24.9	1.88	6.6	2.20	6.2	0.94	2.4	0.95	0.75
3/4	22.4	6.56	23.3	1.89	6.1	2.18	5.8	0.94	2.3	0.93	0.75
11/16	20.6	6.06	21.7	1.89	5.6	2.15	5.5	0.95	2.1	0.90	0.75
5/8	18.9	5.55	20.1	1.90	5.2	2.13	5.1	0.96	1.9	0.88	0.75
9/16	17.1	5.03	18.4	1.91	4.7	2.11	4.7	0.96	1.8	0.86	0.75
1/2	15.3	4.50	16.6	1.92	4.2	2.08	4.3	0.97	1.6	0.83	0.76
7/16	13.5	3.97	14.8	1.93	3.7	2.06	3.8	0.98	1.4	0.81	0.76
3/8	11.7	3.42	12.9	1.94	3.3	2.04	3.3	0.99	1.2	0.79	0.77
5/16	9.8	2.87	10.9	1.95	2.7	2.01	2.9	1.00	1.0	0.76	0.77
7/8	24.2	7.11	16.4	1.52	5.0	1.71	9.2	1.14	3.3	1.21	0.84
13/16	22.7	6.65	15.5	1.53	4.7	1.68	8.7	1.15	3.1	1.18	0.84
3/4	21.1	6.19	14.6	1.54	4.4	1.66	8.2	1.15	2.9	1.16	0.84
11/16	19.5	5.72	13.6	1.54	4.1	1.64	7.7	1.16	2.7	1.14	0.84
5/8	17.8	5.23	12.6	1.55	3.7	1.62	7.1	1.17	2.5	1.12	0.84
9/16	16.2	4.75	11.6	1.56	3.4	1.60	6.6	1.18	2.3	1.10	0.85
1/2	14.5	4.25	10.5	1.57	3.1	1.57	6.0	1.18	2.0	1.07	0.85
7/16	12.8	3.75	9.3	1.58	2.7	1.55	5.3	1.19	1.8	1.05	0.85
3/8	11.0	3.23	8.1	1.59	2.3	1.53	4.7	1.20	1.6	1.03	0.86
7/8	22.7	6.67	15.7	1.53	4.9	1.79	6.2	0.95	2.5	1.04	0.75
13/16	21.3	6.25	14.8	1.54	4.6	1.77	5.9	0.97	2.4	1.02	0.75
3/4	19.8	5.81	13.9	1.55	4.3	1.75	5.6	0.98	2.2	1.00	0.75
11/16	18.3	5.37	13.0	1.56	4.0	1.72	5.2	0.98	2.1	0.97	0.75
5/8	16.8	4.92	12.0	1.56	3.7	1.70	4.8	0.99	1.9	0.95	0.75
9/16	15.2	4.47	11.0	1.57	3.3	1.68	4.4	1.00	1.7	0.93	0.75
1/2	13.6	4.00	10.0	1.58	3.0	1.66	4.0	1.01	1.6	0.91	0.75
7/16	12.0	3.53	8.9	1.59	2.6	1.63	3.6	1.01	1.4	0.88	0.76
3/8	10.4	3.05	7.8	1.60	2.3	1.61	3.2	1.02	1.2	0.86	0.76
5/16	8.7	2.56	6.6	1.61	1.9	1.59	2.7	1.03	1.0	0.84	0.76
13/16	19.9	5.84	14.0	1.55	4.5	1.86	3.7	0.80	1.7	0.86	0.64
3/4	18.5	5.44	13.2	1.55	4.2	1.84	3.5	0.80	1.6	0.84	0.64
11/16	17.1	5.03	12.3	1.56	3.9	1.82	3.3	0.81	1.5	0.82	0.64
5/8	15.7	4.61	11.4	1.57	3.5	1.80	3.1	0.81	1.4	0.80	0.64
9/16	14.3	4.18	10.4	1.58	3.2	1.77	2.8	0.82	1.3	0.77	0.65
1/2	12.8	3.75	9.5	1.59	2.9	1.75	2.6	0.83	1.1	0.75	0.65
7/16	11.3	3.31	8.4	1.60	2.6	1.73	2.3	0.84	1.0	0.73	0.65
3/8	9.8	2.86	7.4	1.61	2.2	1.70	2.0	0.84	0.89	0.70	0.65
5/16	8.2	2.40	6.3	1.61	1.9	1.68	1.8	0.85	0.75	0.68	0.66

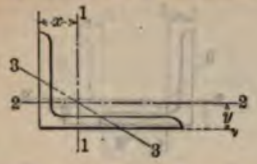
tc" with Table IV, page 354.

ocket Companion, 1913 Edition, Carnegie Steel Company, Pittsburgh, Pa.



Size	Weight per foot	Area of section	Axis 1-1				Axis 2-2		
			I	*r*	*I/c*	*x*	*I*	*r*	*I/c*
in	lb	sq in	in^4	in	in^3	in	in^4	in	in^3
4½×3 ×13/16	18.5	5.43	10.3	1.38	3.6	1.65	3.6	0.81	1.7
4½×3 ×¾	17.3	5.06	9.7	1.39	3.4	1.63	3.4	0.82	1.6
4½×3 ×11/16	16.0	4.68	9.1	1.39	3.1	1.60	3.2	0.83	1.5
4½×3 ×⅝	14.7	4.30	8.4	1.40	2.9	1.58	3.0	0.83	1.4
4½×3 ×9/16	13.3	3.90	7.8	1.41	2.6	1.56	2.8	0.85	1.3
4½×3 ×½	11.9	3.50	7.0	1.42	2.4	1.54	2.5	0.85	1.1
4½×3 ×7/16	10.6	3.09	6.3	1.43	2.1	1.51	2.3	0.85	1.0
4½×3 ×⅜	9.1	2.67	5.5	1.44	1.8	1.49	2.0	0.86	0.88
4½×3 ×5/16	7.7	2.25	4.7	1.44	1.5	1.47	1.7	0.87	0.75
4 ×3½×13/16	18.5	5.43	7.8	1.19	2.9	1.36	5.5	1.01	2.3
4 ×3½×¾	17.3	5.06	7.3	1.20	2.8	1.34	5.2	1.01	2.1
4 ×3½×11/16	16.0	4.68	6.9	1.21	2.6	1.32	4.9	1.02	2.0
4 ×3½×⅝	14.7	4.30	6.4	1.22	2.4	1.29	4.5	1.03	1.8
4 ×3½×9/16	13.3	3.90	5.9	1.23	2.1	1.27	4.2	1.03	1.7
4 ×3½×½	11.9	3.50	5.3	1.23	1.9	1.25	3.8	1.04	1.5
4 ×3½×7/16	10.6	3.09	4.8	1.24	1.7	1.23	3.4	1.05	1.3
4 ×3½×⅜	9.1	2.67	4.2	1.25	1.5	1.21	3.0	1.06	1.2
4 ×3½×5/16	7.7	2.25	3.6	1.26	1.3	1.18	2.6	1.07	1.0
4 ×3 ×13/16	17.1	5.03	7.3	1.21	2.9	1.44	3.5	0.83	1.7
4 ×3 ×¾	16.0	4.69	6.9	1.22	2.7	1.42	3.3	0.84	1.6
4 ×3 ×11/16	14.8	4.34	6.5	1.22	2.5	1.39	3.1	0.84	1.5
4 ×3 ×⅝	13.6	3.98	6.0	1.23	2.3	1.37	2.9	0.85	1.4
4 ×3 ×9/16	12.4	3.62	5.6	1.24	2.1	1.35	2.7	0.86	1.2
4 ×3 ×½	11.1	3.25	5.0	1.25	1.9	1.33	2.4	0.86	1.1
4 ×3 ×7/16	9.8	2.87	4.5	1.25	1.7	1.30	2.2	0.87	1.0
4 ×3 ×⅜	8.5	2.48	4.0	1.26	1.5	1.28	1.9	0.88	0.87
4 ×3 ×5/16	7.2	2.09	3.4	1.27	1.2	1.26	1.7	0.89	0.74
4 ×3 ×¼	5.8	1.69	2.8	1.28	1.0	1.24	1.4	0.89	0.60
3½×3 ×13/16	15.8	4.62	5.0	1.04	2.2	1.23	3.3	0.85	1.7
3½×3 ×¾	14.7	4.31	4.7	1.04	2.1	1.21	3.1	0.85	1.5
3½×3 ×11/16	13.6	4.00	4.4	1.05	1.9	1.19	3.0	0.86	1.4
3½×3 ×⅝	12.5	3.67	4.1	1.06	1.8	1.17	2.8	0.87	1.3
3½×3 ×9/16	11.4	3.34	3.8	1.07	1.6	1.15	2.5	0.87	

XI * (Continued). Properties of Angle-Sections. Unequal Legs

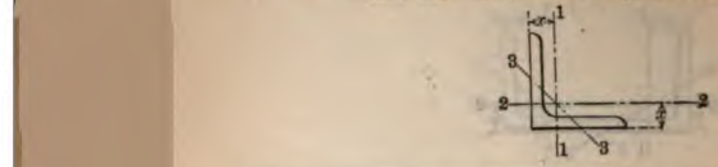


Weight per foot	Area of section	Axis 1-1				Axis 2-2				Axis 3-3
		I	r	I/c	x	I	r	I/c	y	$r_{\min}$
lb	sq in	in⁴	in	in³	in	in⁴	in	in³	in	in
9.5	2.78	2.3	0.91	1.2	1.02	1.4	0.72	0.82	0.77	0.52
8.5	2.50	2.1	0.91	1.0	1.00	1.3	0.72	0.74	0.75	0.52
7.6	2.21	1.9	0.92	0.93	0.98	1.2	0.73	0.66	0.73	0.52
6.6	1.92	1.7	0.93	0.81	0.96	1.0	0.74	0.58	0.71	0.52
5.6	1.62	1.4	0.94	0.69	0.93	0.90	0.74	0.49	0.68	0.53
4.5	1.31	1.2	0.95	0.56	0.91	0.74	0.75	0.40	0.66	0.53
7.7	2.25	1.9	0.92	1.0	1.08	0.67	0.55	0.47	0.58	0.43
6.8	2.00	1.7	0.93	0.89	1.06	0.61	0.55	0.42	0.56	0.43
5.9	1.73	1.5	0.94	0.78	1.04	0.54	0.56	0.37	0.54	0.43
5.0	1.47	1.3	0.95	0.66	1.02	0.47	0.57	0.32	0.52	0.43
4.1	1.19	1.1	0.95	0.54	0.99	0.39	0.57	0.25	0.49	0.43
6.8	2.00	1.1	0.75	0.70	0.88	0.64	0.56	0.46	0.63	0.42
6.1	1.78	1.0	0.76	0.62	0.85	0.58	0.57	0.41	0.60	0.42
5.3	1.55	0.91	0.77	0.55	0.83	0.51	0.58	0.36	0.58	0.42
4.5	1.31	0.79	0.78	0.47	0.81	0.45	0.58	0.31	0.56	0.42
3.62	1.06	0.65	0.78	0.38	0.79	0.37	0.59	0.25	0.54	0.42
2.75	0.81	0.51	0.79	0.29	0.76	0.29	0.60	0.20	0.51	0.43
1.86	0.55	0.35	0.80	0.20	0.74	0.20	0.61	0.13	0.49	0.43
3.92	1.15	0.71	0.79	0.44	0.90	0.19	0.41	0.17	0.40	0.32
3.19	0.94	0.59	0.79	0.36	0.88	0.16	0.41	0.14	0.38	0.32
2.44	0.72	0.46	0.80	0.28	0.85	0.13	0.42	0.11	0.35	0.33
5.6	1.63	0.75	0.68	0.54	0.86	0.26	0.40	0.26	0.48	0.32
5.0	1.45	0.68	0.69	0.48	0.83	0.24	0.41	0.23	0.46	0.32
4.4	1.27	0.61	0.69	0.42	0.81	0.21	0.41	0.20	0.44	0.32
3.66	1.07	0.53	0.70	0.36	0.79	0.19	0.42	0.17	0.42	0.32
2.98	0.88	0.44	0.71	0.30	0.77	0.16	0.42	0.14	0.39	0.32
2.28	0.67	0.34	0.72	0.23	0.75	0.12	0.43	0.11	0.37	0.33
3.99	1.17	0.43	0.61	0.34	0.71	0.21	0.42	0.20	0.46	0.32
3.39	1.00	0.38	0.62	0.29	0.69	0.18	0.42	0.17	0.44	0.32
2.77	0.81	0.32	0.62	0.24	0.66	0.15	0.43	0.14	0.41	0.32
2.12	0.62	0.25	0.63	0.18	0.64	0.12	0.44	0.11	0.39	0.32
1.44	0.42	0.17	0.64	0.13	0.62	0.09	0.45	0.08	0.37	0.33
2.55	0.75	0.30	0.63	0.23	0.71	0.09	0.34	0.10	0.33	0.27
1.96	0.57	0.23	0.64	0.18	0.69	0.07	0.35	0.08	0.31	0.27
2.34	0.69	0.20	0.54	0.18	0.60	0.09	0.35	0.10	0.35	0.27
1.80	0.53	0.16	0.55	0.14	0.58	0.07	0.36	0.08	0.33	0.27
1.23	0.36	0.11	0.56	0.09	0.56	0.05	0.37	0.05	0.31	0.27
2.59	0.76	0.16	0.45	0.16	0.52	0.10	0.35	0.11	0.40	0.26
2.13	0.63	0.13	0.46	0.13	0.50	0.08	0.36	0.09	0.38	0.26
1.64	0.48	0.10	0.46	0.10	0.48	0.07	0.37	0.07	0.35	0.26

te" with *Table IV, page 354.*

ocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XII.* Properties of Angle-Sections. Equal Legs

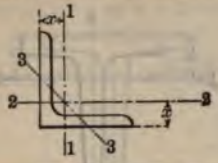


Size	Weight per foot	Area of section	Axis 1-1 and Axis 2-2			
			I	*r*	*I/c*	*x*
in	lb	sq in	in⁴	in	in³	in
8×8×1⅛	56.9	16.73	98.0	2.42	17.5	2.41
8×8×1$\frac{1}{16}$	54.0	15.87	93.5	2.43	16.7	2.39
8×8×1	51.0	15.00	89.0	2.44	15.8	2.37
8×8×$\frac{15}{16}$	48.1	14.12	84.3	2.44	14.9	2.34
8×8×⅞	45.0	13.23	79.6	2.45	14.0	2.32
8×8×$\frac{13}{16}$	42.0	12.34	74.7	2.46	13.1	2.30
8×8×¾	38.9	11.44	69.7	2.47	12.2	2.28
8×8×$\frac{11}{16}$	35.8	10.53	64.6	2.48	11.2	2.25
8×8×⅝	32.7	9.61	59.4	2.49	10.3	2.23
8×8×$\frac{9}{16}$	29.6	8.68	54.1	2.50	9.3	2.21
8×8×½	26.4	7.75	48.6	2.51	8.4	2.19
6×6×1	37.4	11.00	35.5	1.80	8.6	1.86
6×6×$\frac{15}{16}$	35.3	10.37	33.7	1.80	8.1	1.84
6×6×⅞	33.1	9.73	31.9	1.81	7.6	1.82
6×6×$\frac{13}{16}$	31.0	9.09	30.1	1.82	7.2	1.80
6×6×¾	28.7	8.44	28.2	1.83	6.7	1.78
6×6×$\frac{11}{16}$	26.5	7.78	26.2	1.83	6.2	1.75
6×6×⅝	24.2	7.11	24.2	1.84	5.7	1.73
6×6×$\frac{9}{16}$	21.9	6.43	22.1	1.85	5.1	1.71
6×6×½	19.6	5.75	19.9	1.86	4.6	1.68
6×6×$\frac{7}{16}$	17.2	5.06	17.7	1.87	4.1	1.66
6×6×⅜	14.9	4.36	15.4	1.88	3.5	1.64
5×5×1	30.6	9.00	19.6	1.48	5.8	1.61
5×5×$\frac{15}{16}$	28.9	8.50	18.7	1.48	5.5	1.59
5×5×⅞	27.2	7.98	17.8	1.49	5.2	1.57
5×5×$\frac{13}{16}$	25.4	7.47	16.8	1.50	4.9	1.55
5×5×¾	23.6	6.94	15.7	1.50	4.5	1.52
5×5×$\frac{11}{16}$	21.8	6.40	14.7	1.51	4.2	1.50
5×5×⅝	20.0	5.86	13.6	1.52	3.9	1.48
5×5×$\frac{9}{16}$	18.1	5.31	12.4	1.53	3.5	1.46
5×5×½	16.2	4.75	11.3	1.54	3.2	1.43
5×5×$\frac{7}{16}$	14.3	4.18	10.0	1.55	2.8	1.41
5×5×⅜	12.3	3.61	8.7	1.56	2.4	1.39
4×4×$\frac{13}{16}$	19.9	5.84	8.1	1.18	3.0	1.29
4×4×¾	18.5	5.44	7.7	1.19	2.8	1.27
4×4×$\frac{11}{16}$	17.1	5.03	7.2	1.19	2.6	1.25
4×4×⅝	15.7	4.61	6.7	1.20	2.4	1.23
4×4×$\frac{9}{16}$	14.3	4.18	6.1	1.21	2.2	1.21
4×4×½	12.8	3.75	5.6	1.22	2.0	1.18
4×4×$\frac{7}{16}$	11.3	3.31	5.0	1.23	1.8	1.16
4×4×⅜	9.8	2.86	4.4	1.23	1.5	1.14
4×4×$\frac{5}{16}$	8.2	2.40	3.7	1.24	1.3	1.12
4×4×¼	6.6	1.94	3.0	1.25	1.0	1.09

See " Note " with Table IV, page 354.

* From Pocket Companion, 1913 Edition, Carnegie Steel Company,

XII* (Continued). Properties of Angle-Sections. Equal Legs

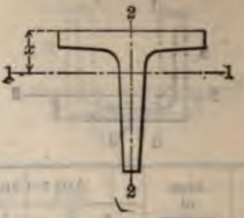


	Weight per foot	Area of section	Axis 1-1 and Axis 2-2				Axis 3-3
			I	r	I/c	x	r_{min}
	lb	sq in	in^4	in	in^3	in	in
13/16	17.1	5.03	5.3	1.02	2.3	1.17	0.67
3/4	16.0	4.69	5.0	1.03	2.1	1.15	0.67
11/16	14.8	4.34	4.7	1.04	2.0	1.12	0.67
5/8	13.6	3.98	4.3	1.04	1.8	1.10	0.68
9/16	12.4	3.62	4.0	1.05	1.6	1.08	0.68
1/2	11.1	3.25	3.6	1.06	1.5	1.06	0.68
7/16	9.8	2.87	3.3	1.07	1.3	1.04	0.68
3/8	8.5	2.48	2.9	1.07	1.2	1.01	0.69
5/16	7.2	2.09	2.5	1.08	0.98	0.99	0.69
1/4	5.8	1.69	2.0	1.09	0.79	0.97	0.69
5/8	11.5	3.36	2.6	0.88	1.3	0.98	0.57
9/16	10.4	3.06	2.4	0.89	1.2	0.95	0.58
1/2	9.4	2.75	2.2	0.90	1.1	0.93	0.58
7/16	8.3	2.43	2.0	0.91	0.95	0.91	0.58
3/8	7.2	2.11	1.8	0.91	0.83	0.89	0.58
5/16	6.1	1.78	1.5	0.92	0.71	0.87	0.59
1/4	4.9	1.44	1.2	0.93	0.58	0.84	0.59
1/2	7.7	2.25	1.2	0.74	0.73	0.81	0.47
7/16	6.8	2.00	1.1	0.75	0.65	0.78	0.48
3/8	5.9	1.73	0.98	0.75	0.57	0.76	0.48
5/16	5.0	1.47	0.85	0.76	0.48	0.74	0.49
1/4	4.1	1.19	0.70	0.77	0.39	0.72	0.49
3/16	3.07	0.90	0.55	0.78	0.30	0.69	0.49
1/8	2.08	0.61	0.38	0.79	0.20	0.67	0.50
7/16	5.3	1.56	0.54	0.59	0.40	0.66	0.39
3/8	4.7	1.36	0.48	0.59	0.35	0.64	0.39
5/16	3.92	1.15	0.42	0.60	0.30	0.61	0.39
1/4	3.19	0.94	0.35	0.61	0.25	0.59	0.39
3/16	2.44	0.71	0.28	0.62	0.19	0.57	0.40
1/8	1.65	0.48	0.19	0.63	0.13	0.55	0.40
7/16	4.6	1.34	0.35	0.51	0.30	0.59	0.33
3/8	3.99	1.17	0.31	0.51	0.26	0.57	0.34
5/16	3.39	1.00	0.27	0.52	0.23	0.55	0.34
1/4	2.77	0.81	0.23	0.53	0.19	0.53	0.34
3/16	2.12	0.62	0.18	0.54	0.14	0.51	0.35
1/8	1.44	0.42	0.13	0.55	0.10	0.48	0.35
3/8	3.35	0.98	0.19	0.44	0.19	0.51	0.29
5/16	2.86	0.84	0.16	0.44	0.16	0.49	0.29
1/4	2.34	0.69	0.14	0.45	0.13	0.47	0.29
3/16	1.80	0.53	0.11	0.46	0.10	0.44	0.29
1/8	1.23	0.36	0.08	0.46	0.07	0.42	0.30
5/16	2.33	0.68	0.09	0.36	0.11	0.42	0.24
1/4	1.92	0.56	0.08	0.37	0.09	0.40	0.24
3/16	1.48	0.43	0.06	0.38	0.07	0.38	0.24
1/8	1.01	0.30	0.04	0.38	0.05	0.35	0.25
1/4	1.49	0.44	0.04	0.29	0.06	0.34	0.19
3/16	1.16	0.34	0.03	0.30	0.04	0.32	0.19
1/8	0.80	0.23	0.02	0.31	0.03	0.30	0.19

e " with Table IV, page 354.

cket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XIII.* Properties of ⊤ Sections. Flange and Stem Eq

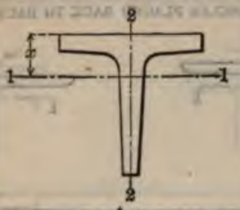


Size				Weight per foot	Area of section	Axis 1-1				Ax
Flange	Stem	Minimum thickness				I	r	I/c	x	I
		Flange	Stem							
in	in	in	in	lb	sq in	in⁴	in	in³	in	in⁴
4	4	½	½	13.5	3.97	5.7	1.20	2.0	1.18	2.8
4	4	⅜	⅜	10.5	3.09	4.5	1.21	1.6	1.13	2.1
3½	3½	½	½	11.7	3.44	3.7	1.04	1.5	1.05	1.9
3½	3½	⅜	⅜	9.2	2.68	3.0	1.05	1.2	1.01	1.4
3	3	½	½	9.9	2.91	2.3	0.88	1.1	0.93	1.2
3	3	7/16	7/16	8.9	2.59	2.1	0.89	0.98	0.91	1.0
3	3	⅜	⅜	7.8	2.27	1.8	0.90	0.86	0.88	0.90
3	3	5/16	5/16	6.7	1.95	1.6	0.90	0.74	0.86	0.75
2½	2½	⅜	⅜	6.4	1.87	1.0	0.74	0.59	0.76	0.52
2½	2½	5/16	5/16	5.5	1.60	0.88	0.74	0.50	0.74	0.44
2¼	2¼	5/16	5/16	4.9	1.43	0.65	0.67	0.41	0.68	0.33
2¼	2¼	¼	¼	4.1	1.19	0.52	0.66	0.32	0.65	0.25
2	2	5/16	5/16	4.3	1.26	0.44	0.59	0.31	0.61	0.23
2	2	¼	¼	3.56	1.05	0.37	0.59	0.26	0.59	0.18
1¾	1¾	¼	¼	3.09	0.91	0.23	0.51	0.19	0.54	0.12
1½	1½	¼	¼	2.47	0.73	0.15	0.45	0.14	0.47	0.08
1½	1½	3/16	3/16	1.94	0.57	0.11	0.45	0.11	0.44	0.06
1¼	1¼	¼	¼	2.02	0.59	0.08	0.37	0.10	0.40	0.05
1¼	1¼	3/16	3/16	1.59	0.47	0.06	0.37	0.07	0.38	0.03
1	1	3/16	3/16	1.25	0.37	0.03	0.29	0.05	0.32	0.02
1	1	⅛	⅛	0.89	0.26	0.02	0.30	0.03	0.29	0.01

See "Note" with Table IV, page 354.

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitts

Table XIV.* Properties of T Sections. Flange and Stem Unequal



Size			Weight per foot	Area of section	Axis 1-1				Axis 2-2		
Stem	Minimum thickness				*I*	*r*	*I/c*	*x*	*I*	*r*	*I/c*
	Flange	Stem									
in	in	in	lb	sq in	in^4	in	in^3	in	in^4	in	in^3
3	1/2	13/32	13.4	3.93	2.4	0.78	1.1	0.73	5.4	1.17	2.2
2½	3/8	7/16	10.9	3.18	1.5	0.68	0.78	0.63	4.1	1.14	1.6
3½	7/16	11/16	15.7	4.60	5.1	1.05	2.1	1.11	3.7	0.90	1.7
3	3/8	3/8	9.8	2.88	2.1	0.84	0.91	0.74	3.0	1.02	1.3
3	5/16	5/16	8.4	2.46	1.8	0.85	0.78	0.71	2.5	1.01	1.1
2½	3/8	3/8	9.2	2.68	1.2	0.67	0.63	0.59	3.0	1.05	1.3
2½	5/16	5/16	7.8	2.29	1.0	0.68	0.54	0.57	2.5	1.05	1.1
5	1/2	1/2	15.3	4.50	10.8	1.55	3.1	1.56	2.8	0.79	1.4
5	3/8	3/8	11.9	3.49	8.5	1.56	2.4	1.51	2.1	0.78	1.1
4½	1/2	1/2	14.4	4.23	7.9	1.37	2.5	1.37	2.8	0.81	1.4
4½	3/8	3/8	11.2	3.29	6.3	1.39	2.0	1.31	2.1	0.80	1.1
3	3/8	3/8	9.2	2.68	2.0	0.86	0.90	0.78	2.1	0.89	1.1
3	5/16	5/16	7.8	2.29	1.7	0.87	0.77	0.75	1.8	0.88	0.88
2½	3/8	3/8	8.5	2.48	1.2	0.69	0.62	0.62	2.1	0.92	1.0
2½	5/16	5/16	7.2	2.12	1.0	0.69	0.53	0.60	1.8	0.91	0.88
2	3/8	3/8	7.8	2.27	0.60	0.52	0.40	0.48	2.1	0.96	1.1
2	7/16	5/16	6.7	1.95	0.53	0.52	0.34	0.46	1.8	0.95	0.88
4	1/2	1/2	12.6	3.70	5.5	1.21	2.0	1.24	1.9	0.72	1.1
4	3/8	3/8	9.8	2.88	4.3	1.23	1.5	1.19	1.4	0.70	0.81
3	1/2	1/2	10.8	3.17	2.4	0.87	1.1	0.88	1.9	0.77	1.1
3	3/8	3/8	8.5	2.48	1.9	0.88	0.89	0.83	1.4	0.75	0.81
3	5/16	3/8	7.5	2.20	1.8	0.91	0.85	0.85	1.2	0.74	0.68
4	1/2	1/2	11.7	3.44	5.2	1.23	1.9	1.32	1.2	0.59	0.81
4	7/16	7/16	10.5	3.06	4.7	1.23	1.7	1.29	1.1	0.59	0.70
4	3/8	3/8	9.2	2.68	4.1	1.24	1.5	1.27	0.90	0.58	0.60
3½	1/2	1/2	10.8	3.17	3.5	1.06	1.5	1.12	1.2	0.62	0.80
3½	7/16	7/16	9.7	2.83	3.2	1.06	1.3	1.10	1.0	0.60	0.69
3½	3/8	3/8	8.5	2.48	2.8	1.07	1.2	1.07	0.93	0.61	0.62
2½	3/8	3/8	7.1	2.07	1.1	0.72	0.60	0.71	0.89	0.66	0.59
2½	5/16	5/16	6.1	1.77	0.94	0.73	0.52	0.68	0.75	0.65	0.50
3	3/8	3/8	7.1	2.07	1.7	0.91	0.84	0.95	0.53	0.51	0.42
3	5/16	5/16	6.1	1.77	1.5	0.92	0.72	0.92	0.44	0.50	0.35
1¾	3/16	3/16	2.87	0.84	0.08	0.31	0.09	0.32	0.29	0.58	0.23
1½	1/4	1/4	3.09	0.91	0.16	0.42	0.15	0.42	0.18	0.45	0.18
2	3/16	3/16	2.45	0.72	0.27	0.61	0.19	0.63	0.06	0.92	0.08
1¼	1/8	1/8	1.25	0.37	0.05	0.37	0.05	0.33	0.04	0.32	0.05
5/8	No. 9	1/8	0.88	0.26	0.01	0.16	0.01	0.16	0.02	0.31	0.04

"Note" with *Table IV, page 354.*

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Single angle		Two angles	Radii of gyration, r, in inches				
				Axis 2–2			
Size, in	Weight per foot, lb	Area, sq in	Axis 1–1	In contact	¼-in apart	⅜-in apart	½-in apart
8 × 8 × 1⅛	56.9	33.46	2.42	3.42	3.51	3.55	3.60
13/16	42.0	24.68	2.46	3.37	3.46	3.50	3.55
½	26.4	15.51	2.51	3.33	3.41	3.45	3.50
6 × 6 × 1	37.4	22.00	1.80	2.59	2.68	2.72	2.77
11/16	26.5	15.56	1.83	2.54	2.63	2.67	2.71
⅜	14.9	8.72	1.88	2.49	2.58	2.62	2.66
5 × 5 × 1	30.6	18.00	1.48	2.19	2.28	2.33	2.38
11/16	21.8	12.80	1.51	2.13	2.22	2.26	2.31
⅜	12.3	7.22	1.56	2.09	2.17	2.21	2.26
4 × 4 × 13/16	19.9	11.68	1.18	1.75	1.85	1.89	1.94
¼	6.6	3.88	1.25	1.66	1.75	1.79	1.84
3½ × 3½ × 13/16	17.1	10.06	1.02	1.55	1.65	1.70	1.75
¼	5.8	3.38	1.09	1.46	1.55	1.59	1.64
3 × 3 × ⅝	11.5	6.72	0.88	1.32	1.41	1.46	1.51
¼	4.9	2.88	0.93	1.25	1.34	1.38	1.43
2½ × 2½ × ½	7.7	4.50	0.74	1.09	1.19	1.24	1.29
¼	4.1	2.38	0.77	1.05	1.14	1.19	1.24
2 × 2 × 7/16	5.3	3.12	0.59	0.88	0.98	1.03	1.08
¼	3.19	1.88	0.61	0.85	0.94	0.99	1.04

This table and the two following are employed in computing the safe resi compressive stress of two angles, back to back, used as struts or as the com chords of roof-trusses, etc., by the following rule:

Obtain from the compression-formula in use the allowed stress per sq corresponding to the ratio of slenderness of the section, and multiply that the area. The result will be the allowable compressive stress.

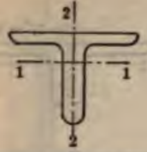
Example 1. Section given. Required the safe load in compression, as pe $S = 19\,000 - 100\,l/r$, on a strut composed of two angles, 4 by 4 by ¼ in, back with an unsupported length of 9 ft.

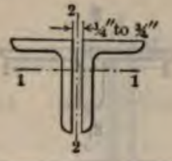
Area of section, $A = 3.88$ sq in; least radius of gyration, $r = 1.25$ in.

Ratio of slenderness, $l/r = 9 \times 12 \div 1.25 = 86.4$.

XVI.* Properties of Double-Angle Sections. Long Legs Vertical

ANGLES PLACED BACK TO BACK



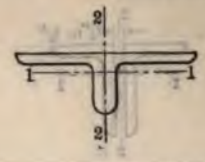


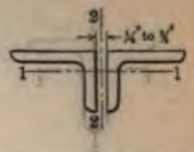
angle		Two angles	Radii of gyration, r, in inches					
				Axis 2-2				
	Weight per foot, lb	Area, sq in	Axis 1-1	In contact	¼-in apart	⅜-in apart	½-in apart	¾-in apart
1	44.2	26.00	2.49	2.39	2.48	2.52	2.57	2.66
¾	33.8	19.88	2.53	2.35	2.44	2.48	2.52	2.61
7/16	20.2	11.86	2.57	2.31	2.39	2.43	2.48	2.56
1	35.7	21.00	2.51	1.26	1.35	1.40	1.45	1.55
¾	27.5	16.12	2.55	1.20	1.29	1.34	1.39	1.49
7/16	16.5	9.68	2.59	1.15	1.23	1.28	1.32	1.41
1	32.3	19.00	2.19	1.31	1.40	1.45	1.50	1.60
11/16	23.0	13.50	2.23	1.25	1.34	1.39	1.44	1.53
⅜	13.0	7.60	2.27	1.20	1.28	1.33	1.37	1.46
1	30.6	18.00	1.85	1.60	1.69	1.74	1.79	1.89
11/16	21.8	12.80	1.89	1.55	1.63	1.68	1.73	1.82
⅜	12.3	7.22	1.93	1.50	1.58	1.62	1.67	1.76
1	28.9	17.00	1.85	1.37	1.47	1.51	1.56	1.66
11/16	20.6	12.12	1.89	1.31	1.41	1.45	1.49	1.60
5/16	9.8	5.74	1.95	1.25	1.33	1.37	1.42	1.50
⅞	24.2	14.22	1.52	1.66	1.76	1.80	1.85	1.95
⅜	11.0	6.46	1.59	1.58	1.66	1.70	1.75	1.85
⅞	22.7	13.34	1.53	1.42	1.51	1.56	1.61	1.71
5/16	8.7	5.12	1.61	1.33	1.41	1.45	1.50	1.59
13/16	19.9	11.68	1.55	1.18	1.27	1.32	1.37	1.47
5/16	8.2	4.80	1.61	1.09	1.17	1.22	1.26	1.35
13/16	18.5	10.86	1.38	1.21	1.31	1.36	1.41	1.51
5/16	7.7	4.50	1.44	1.13	1.22	1.26	1.30	1.40
13/16	18.5	10.86	1.19	1.50	1.59	1.64	1.69	1.79
5/16	7.7	4.50	1.26	1.42	1.51	1.55	1.60	1.69
13/16	17.1	10.06	1.21	1.25	1.35	1.40	1.45	1.55
¼	5.8	3.38	1.28	1.16	1.24	1.28	1.33	1.43
13/16	15.8	9.24	1.04	1.30	1.40	1.45	1.50	1.60
¼	5.4	3.12	1.11	1.20	1.29	1.34	1.38	1.48
11/16	12.5	7.30	1.06	1.03	1.13	1.18	1.23	1.33
¼	4.9	2.88	1.12	0.95	1.04	1.09	1.13	1.23
9/16	9.5	5.56	0.91	1.05	1.15	1.20	1.25	1.35
¼	4.5	2.64	0.95	1.00	1.09	1.13	1.18	1.28
½	7.7	4.50	0.92	0.80	0.89	0.94	1.00	1.10
¼	4.1	2.38	0.95	0.74	0.84	0.88	0.93	1.03
½	6.8	4.00	0.75	0.84	0.94	0.99	1.04	1.15
¼	3.62	2.12	0.78	0.80	0.89	0.93	0.98	1.08

ocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XVII.* Properties of Double-Angle Sections. Short Legs Vertical

ANGLES PLACED BACK TO BACK



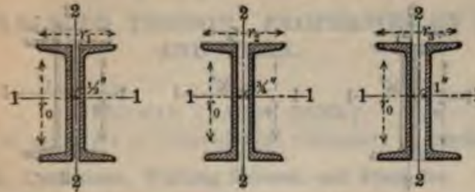


Single angle		Two angles	Radii of gyration, r, in inches				
				Axis 2-2			
Size, in	Weight per foot, lb	Area, sq in	Axis 1-1	In contact	¼-in apart	⅜-in apart	½-in apart
8 ×6 ×1	44.2	26.00	1.73	3.64	3.73	3.78	3.83
¾	33.8	19.88	1.76	3.60	3.69	3.73	3.78
7/16	20.2	11.86	1.80	3.55	3.64	3.68	3.73
8 ×3½×1	35.7	21.00	0.86	4.04	4.14	4.19	4.24
¾	27.5	16.12	0.88	3.99	4.09	4.13	4.18
7/16	16.5	9.68	0.92	3.93	4.02	4.07	4.12
7 ×3½×1	32.3	19.00	0.89	3.48	3.58	3.63	3.68
11/16	23.0	13.50	0.92	3.42	3.52	3.57	3.62
⅜	13.0	7.60	0.96	3.36	3.46	3.50	3.55
6 ×4 ×1	30.6	18.00	1.09	2.85	2.95	2.99	3.04
11/16	21.8	12.80	1.13	2.79	2.89	2.93	2.98
⅜	12.3	7.22	1.17	2.74	2.83	2.87	2.92
6 ×3½×1	28.9	17.00	0.92	2.92	3.02	3.07	3.12
11/16	20.6	12.12	0.95	2.87	2.96	3.01	3.06
5/16	9.8	5.74	1.00	2.81	2.90	2.95	3.00
5 ×4 ×⅞	24.2	14.22	1.14	2.29	2.38	2.43	2.48
⅜	11.0	6.46	1.20	2.20	2.29	2.34	2.38
5 ×3½×⅞	22.7	13.34	0.96	2.36	2.45	2.50	2.55
5/16	8.7	5.12	1.03	2.26	2.35	2.39	2.44
5 ×3 ×13/16	19.9	11.68	0.80	2.42	2.52	2.57	2.62
5/16	8.2	4.80	0.85	2.33	2.42	2.47	2.52
4½×3 ×13/16	18.5	10.86	0.81	2.15	2.25	2.30	2.35
5/16	7.7	4.50	0.87	2.06	2.15	2.20	2.25
4 ×3½×13/16	18.5	10.86	1.01	1.81	1.91	1.96	2.01
5/16	7.7	4.50	1.07	1.73	1.81	1.86	1.91
4 ×3 ×13/16	17.1	10.06	0.83	1.88	1.98	2.03	2.08
¼	5.8	3.38	0.89	1.78	1.87	1.92	1.96
3½×3 ×13/16	15.8	9.24	0.85	1.61	1.71	1.76	1.81
¼	5.4	3.12	0.91	1.52	1.61	1.65	1.70
3½×2½×11/16	12.5	7.30	0.69	1.66	1.75	1.80	1.86
¼	4.9	2.88	0.74	1.58	1.67	1.71	1.76
3 ×2½×9/16	9.5	5.56	0.72	1.37	1.46	1.51	1.56
¼	4.5	2.64	0.75	1.31	1.40	1.45	1.50
3 ×2 ×½	7.7	4.50	0.55	1.42	1.52	1.57	1.62
¼	4.1	2.38	0.57	1.38	1.47	1.52	1.57
2½×2 ×½	6.8	4.00	0.56	1.15	1.25	1.30	1.35
¼	3.62	2.12	0.59	1.11	1.20	1.25	1.30

† From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

Table XVIII. Properties of Double-Channel Sections

STANDARD CHANNELS PLACED BACK TO BACK

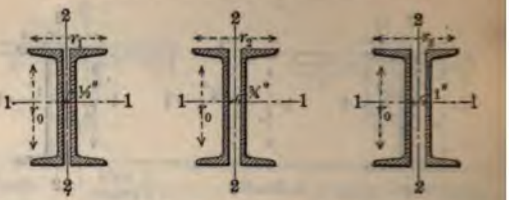


radii of gyration given correspond to directions indicated by the arrow-heads

th,	Thickness of web, in	Weight per foot of one channel, lb	Area of two channels, sq in	Radii of gyration, r, in inches			
				Axis 1-1	Axis 2-2		
					½-in apart	¾-in apart	1-in apart
	0.40	33.00	19.80	5.62	1.38	1.48	1.58
	0.43	35.00	20.58	5.58	1.38	1.47	1.57
	0.52	40.00	23.52	5.43	1.37	1.46	1.56
	0.62	45.00	26.48	5.32	1.37	1.45	1.56
	0.72	50.00	29.42	5.23	1.37	1.46	1.56
	0.82	55.00	32.36	5.16	1.38	1.47	1.58
	0.28	20.50	12.06	4.61	1.24	1.34	1.44
	0.39	25.00	14.70	4.43	1.21	1.31	1.41
	0.51	30.00	17.64	4.28	1.20	1.30	1.40
	0.64	35.00	20.58	4.17	1.21	1.31	1.41
	1.76	40.00	23.52	0.09	1.23	1.32	1.43
	0.24	15.00	8.92	3.87	1.14	1.24	1.34
	0.38	20.00	11.76	3.66	1.10	1.20	1.31
	0.53	25.00	14.70	3.52	1.10	1.20	1.31
	0.68	30.00	17.64	3.42	1.12	1.22	1.33
	0.82	35.00	20.58	3.35	1.16	1.26	1.37
	0.23	13.25	7.78	3.49	1.09	1.19	1.29
	0.29	15.00	8.82	3.40	1.07	1.17	1.28
	0.45	20.00	11.76	3.21	1.05	1.15	1.26
	0.62	25.00	14.70	3.10	1.07	1.17	1.28

Table XVIII (Continued). Properties of Double-Channel Sections

STANDARD CHANNELS PLACED BACK TO BACK



The radii of gyration given correspond to directions indicated by the arro

Depth, in	Thickness of web, in	Weight per foot of one channel, lb	Area of two channels, sq in	Radii of gyration, r, in inc		
				Axis 1-1	Axis 2-2	
					½-in apart	¾-in apart
8	0.22	11.25	6.70	3.11	1.04	1.14
8	0.31	13.75	8.08	2.98	1.04	1.14
8	0.40	16.25	9.56	2.89	1.03	1.14
8	0.49	18.75	11.02	2.82	1.03	1.14
*8	0.58	21.25	12.50	2.77	1.03	1.14
7	0.21	9.75	5.70	2.72	0.99	1.09
7	0.32	12.25	7.20	2.59	0.99	1.09
7	0.42	14.75	8.68	2.50	0.99	1.10
7	0.53	17.25	10.14	2.44	1.00	1.10
7	0.63	19.75	11.62	2.39	1.00	1.10
6	0.20	8.00	4.76	2.34	0.94	1.05
6	0.32	10.50	6.18	2.21	0.94	1.05
6	0.44	13.00	7.64	2.13	0.95	1.06
6	0.56	15.50	9.12	2.07	0.95	1.06
5	0.19	6.50	3.90	1.95	0.89	1.00
5	0.33	9.00	5.30	1.83	0.90	1.00
5	0.48	11.50	6.76	1.75	0.91	1.01
4	0.18	5.25	3.10	1.56	0.84	0.95
4	0.25	6.25	3.68	1.51	0.84	0.95
4	0.32	7.25	4.26	1.46	0.84	0.95
3	0.17	4.00	2.38	1.17	0.80	0.91
3	0.26	5.00	2.94	1.12	0.81	0.92
3	0.36	6.00	3.52	1.08	0.83	0.93

CHAPTER XI

TANCE TO TENSION. PROPERTIES OF IRON AND STEEL

BY
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1. Definitions, Working Stresses and Examples

timate Tensile Strength of a material is the amount of internal
:h a section one square inch in area is capable of exerting against an
xial force. It is the UNIT STRESS OR INTENSITY OF STRESS, expressed
per square inch, which the material can withstand. It is often called
ATE STRENGTH OR ULTIMATE STRESS of the material. Its value for
ial depends on the tenacity of the fibers or the cohesion of the particles
he material is composed.

al Force is one which acts uniformly over the section of a prismatic
hat the resultant of the distributed forces coincides with the axis of
Hence the total AXIAL FORCE which any cross-section of a body will
e product of the ultimate strength of the material and the area of
section, in square inches.

'orking Stress. The ultimate strength of different building materials
found by pulling apart bars of known dimensions and dividing the
load each sustained by the area of the bar before testing. This ulti-
gth, however, must not be used to proportion the size of members
res, because of variations in material, hidden defects and imperfect
hip; and, especially, because of indefiniteness as to the maximum load
be imposed on the structure. To provide safety against the rupture
ber and the consequent failure of the structure from any of these
proportions of the members must be based on SAFE WORKING STRESSES
usually some fractional part of the ultimate strength found by
t to provide proper security against failure.

ctor of Safety is the ratio of the ultimate strength to this safe
tress for that material. Its value ranges generally from 2 to 10,
upon the nature of the material and the service to which it is applied.

'orking Stress in Tension. Table I gives these values for various
aterials. The total SAFE LOAD that may be applied to a piece of
uniform section is found by multiplying the cross-section of the piece,
inches, by the safe working stress opposite the name of the material
he piece is composed.

P = the safe load in lb,
S_t = the allowable safe working stress in tension,
b = the width of a rectangular bar,
h = the depth of a rectangular bar,
d = the diameter of a round bar,

s, for a rectangular bar,

$$P = bhS_t \qquad (1)$$

und bar,

$$P = 0.7854\, d^2 S_t \qquad (2)$$

The area of cross-section to support a load P is, for a rectangular ba

$$A = \frac{P}{S_t}$$

and for a round bar

$$d = \sqrt{\frac{P}{0.7854\, S_t}}$$

Table I. Safe Working Stress in Tension for Building Materia

Material	Safe st lb per s (S
Cast iron (New York City)	5 00
Wrought iron	12 00
Steel, medium	16 00
Chestnut	85
Hemlock (New York City)	60
Pine, long-leaf yellow	1 20
Pine, Douglas fir	80
Pine, Norway	80
Pine, white	70
Redwood	70
Spruce	80

Except where noted, the above values are in accordance with the recommend the Association of Railway Superintendents.

Example 1. What size of medium-steel angle should be used to s tensile force of 64 000 lb?

Answer. By formula (3),

$$\text{the net sectional area} = \frac{64\ 000}{16\ 000} = 4.00 \text{ sq in}$$

From the Table of the Properties of Angles (Chapter X) we find tha 4 by ⅝-in angle has an area of 4.61 sq in, which is to be reduced by a ⅞ for a ¾-in rivet, leaving $4.61 - (\frac{7}{8} \times \frac{5}{8}) = 4.06$ sq in, net area. This is in excess of the required amount.

The SAFE LOAD for angles commonly used in roof-trusses is given in T and the REDUCTION IN SECTIONAL AREA caused by rivet-holes, in Ta this chapter, and in Table I, Chapter XX. See, also, Chapter XII, pa paragraph on Punching Rivet-Holes.

Example 2. What size of white-pine tie-beam should be used to s tensile force of 60 000 lb?

Answer. By formula (3),

$$\text{the net sectional area} = \frac{60\ 000}{700} = 85.7 \text{ sq in}$$

If the depth is taken at 12 in, the net width must be $\frac{85.7}{12} = 7.2$ in. A must be made for the increase in tension on the lower side of the bean its own weight, and also for any cutting that may be necessary in ma connections or holes for truss-rods. If there is a 2-in hole through th

sphorus, sulphur and manganese. It is made from pig iron and
or mill-scale, in a reverberatory furnace consisting of a firebox, a
orking-chamber, and the necessary dampers and flues. The impur-
noved from the iron at different stages in the process, silicon and
during the melting-down stage, part of the phosphorus and sulphur
clearing-stage and the carbon and remainder of the phosphorus and
ing the boiling-stage. The iron is then in a pasty condition ready
gh stirring by the workman, who collects it into balls of about 80 lb
takes it to a squeezer or forge where the greater part of the slag is
It is then rolled out into MUCK-BARS. These bars are cut into pieces
iled into bundles suited to the size of the finished bar. The piles
nd rolled again. The rolling reduces the amount of slag and makes
l denser. The process of rerolling may be repeated a number of
duce double or triple-refined MERCHANT-BAR IRON.

earance of Wrought Iron is very much like that of steel. It may
shed from steel by nicking one side of the bar and bending it away
k. Iron will split along the slag-laminations and show the COARSELY
ture of the material; while steel will bend or rupture at the nick
itting, any fracture being FINELY FIBROUS or CRYSTALLINE. When
a tension-test wrought iron shows a dark fibrous fracture. If the
grooved before testing or broken in impact the fracture will be
stalline.

Wrought iron is more easily welded than steel because the work
omplished through a wider range of temperature than with steel.
develop the full strength of the bar, but tests on hand-forged welds
e-bars reported by Kirkaldy gave average values of about 60% of
of the bar.

rought iron is no longer used for the manufacture of structural
as angles, channels and beams, its use for structural work being
limited to bars, rods and bolts. It can be worked more easily than
ading-machines; and on this account, unless steel is specified, some

Elongation, percentage in 8 in.	[illegible]	[illegible]	[illegible]

4. Nicking-Test. When the specimen is slightly and evenly nic
side and bent back through an angle of 180° it shall show the followin

(b) Merchant iron, Grade A, a long, clean, silky fiber, free from
or any coarse crystalline spots. A few fine crystalline spots may b
provided they do not in the aggregate exceed 10% of the sectional
bar.

(c) Merchant iron, Grade B, a generally fibrous fracture, free
crystalline spots. Not over 10% of the fractured surface shall be

(d) Merchant iron, Grade C, a generally fibrous fracture, free
crystalline spots. Not over 15% of the fractured surface shall be

5. Cold Bending-Test. A specimen cut from the bar as roll
bent through an angle of 180° by pressure or by a succession of
The classes of Merchant iron shall conform to the following bendin

(f) Merchant iron, Grade A, shall bend, cold, 180° flat on its
fracture on the outside of the bent portion.

(g) Merchant iron, Grade B, shall bend, cold, 180° around a dian
to the thickness of the specimen tested, without fracture on the ou
bent portion.

(h) Merchant iron, Grade C, shall bend, cold, around a diamet
twice the thickness of the specimen tested, without fracture on the
the bent portion.

6. Hot Bending-Test. Specimens cut from the bar as rolled,
bright-red heat, shall be bent through an angle of 180° by pressure o
cession of light blows, without hammering directly on the bend.
of Merchant iron shall conform to the following hot-bending tests:

(j) Merchant iron, Grade A, shall bend flat on itself, without sho
or flaws. A similar specimen heated to a yellow heat and suddenl
in water the temperature of which is between 80° and 90° F., shall
out hammering on the bend, 180° flat on itself, without showing crac
A similar specimen heated to a bright-red heat, shall be split at the e
part bent back through an angle of 180°. It shall also be quench

on has been defined as a saturated solution of carbon in iron, the tent varying from 1½ to 4% according to the other impurities con- is hard, brittle, non-malleable and very fluid when melted, so that lapted for casting into complex forms.

cture. It is produced in the blast-furnace, which is essentially a ctory-lined stack, with a valve-charging device at the top, tuyeres in the lower part for the introduction of the air-blast, and a hearth om with a tap-hole for the periodic withdrawal of the iron and slag. CE-IRON is cast into PIGS about 3 ft long and weighing about 100 lb NDRY-CASTINGS are made from PIG IRON and SCRAP melted in a cupola into green-sand molds. The charge is made up of different quanti- different grades of pig so as to control the physical properties of the incipally through control of the silicon-content.

nce. CASTINGS have a gray or white fracture according to the condi- contained carbon, the gray fracture indicating graphitic or separated the white the combined carbon. GRAY IRON is softer and tougher fied for ordinary CASTINGS.

. Cast iron does not have a definite ELASTIC LIMIT. A relatively will produce some permanent deformation. Its ULTIMATE TENSILE aries from 15 000 to 20 000 lb per sq in; and in some iron is as high b per sq in. Its COMPRESSIVE STRENGTH varies over a wide range, r sq in being a fair average value.

Castings are liable to several common DEFECTS the chief of which les due to the formation of steam from the damp molds, sand-holes laced sand, rough surfaces, cold shuts due to chilling of the iron to fill the parts of the mold, shrinkage-cracks due to uneven cooling ngs in parts of different thickness. In cored castings, also, the walls ly of variable thickness because of the shifting of the cores. This is requent in case of hollow columns cast in a horizontal position. these defects and on account of the low ULTIMATE STRENGTH, cast never be used where it is subjected to any great tensile stress.

tions* for Cast Iron. The specifications of the American Society

For medium castings, not under 2 900 lb.
For heavy castings, not under 3 300 lb.

In no case shall the DEFLECTION be under 0.10 in.

TENSION-TEST. Where specified this shall be:

For light castings, not less than 18 000 lb per sq in.
For medium castings, not less than 21 000 lb per sq in.
For heavy castings, not less than 24 000 lb per sq in.

The specifications give explicit directions for casting the ARBIT
which is 1¼ in in diameter and 15 in long. Two of these are c
twenty tons of castings. One of each pair must fulfill the requirem
mit acceptance of the castings. The bar is loaded at the middle at
will cause a 0.10-in deflection in from twenty to forty seconds. T
test is not recommended.

11. CASTINGS shall be true to pattern, free from cracks, flaws ar
shrinkage. In other respects they shall conform to whatever poi
specially agreed upon.

4. Steel

Steel is a mixture of compounds of iron and carbon with small c
other elements, including manganese, phosphorus, sulphur, silicon,
carbon-content controls the hardness and strength of the steel.
0.10% of carbon is present in the soft steels, which have most of
teristics of wrought iron; while steel with more than 0.40% carbo
of being tempered, cannot be welded and is very much stronger.
acts as a cleanser during the process of manufacture, and increase
ability of the steel. Phosphorus and sulphur are harmful in their e
phorus making steel brittle under sudden loading and sulphur making
or brittle when heated.

Manufacture. STRUCTURAL STEEL is manufactured by the BE
the OPEN-HEARTH PROCESSES. In the first, molten cast iron is cha
Bessemer converter, an air-blast is driven through the charge from
in the false bottom of the converter and the silicon, sulphur and ca

The material is more uniform and consequently more dependable an BESSEMER STEEL. OPEN-HEARTH STEEL is used for most struc-

us may be removed by the basic process as in case of Bessemer running low in phosphorus are generally used in America so that ocess is little employed here.

ct of Carbon and Phosphorus on the STATIC STRENGTH of steel s of carbon included in structural steel is an increase in strength of lb per sq in for each 0.01% increase in either element. Cunning-ula

$$S_t = 40\,000 + 100\,000\,(C + P)$$

proximate relation between the strength and the chemical composi-d P are respectively the amounts of carbon and phosphorus ex-ercentage. For example, the ULTIMATE STRENGTH of a steel having on and 0.07% phosphorus is, approximately,

$$_t = 40\,000 + 100\,000\,(0.15 + 0.07) = 62\,000 \text{ lb per sq in}$$

centage of Elongation decreases as the carbon-content and ULTI-GTH increase. An approximate relation being

$$\text{percentage of elongation} = \frac{1\,400\,000}{S_t}$$

TAL ELONGATION of a ruptured specimen is due to the local stretch-oint of rupture and the uniform elongation over the whole gauge- necessary to report the gauge-length when reporting this result. CAL ELONGATION is the same for a 2 or an 8-in length, the PERCENT-GATION for the same material, tested on a 2-in gauge-length, is greater sured on an 8-in length.

tic Behavior of a specimen of steel loaded to rupture is best shown -STRAIN DIAGRAM on which the stresses are plotted as vertical ordi-e elongations or strains as abscissas, as in Fig. 1. Five significant hown:

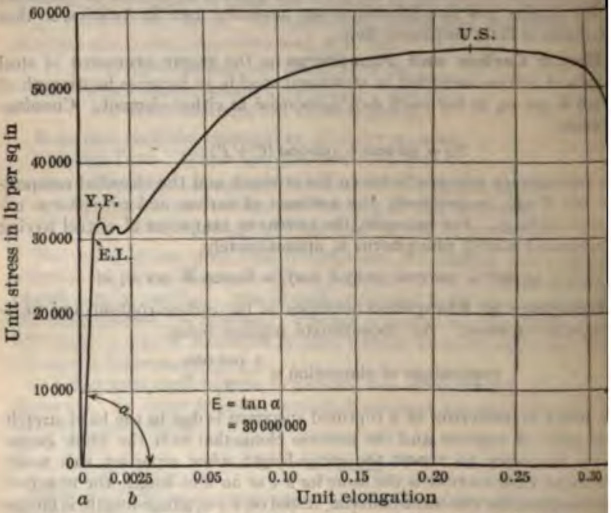
Modulus of Elasticity (E). The relation between the stress and the ngation is called the MODULUS OF ELASTICITY. It is equal to the livided by the unit strain or deformation and is represented graph- tangent of the angle of the initial line with the horizontal. Its el for tension is about 30 000 000 lb per sq in.

Elastic Limit ($E.L.$) is that unit stress beyond which the ratio of ain ceases to be constant, or beyond which the curve ceases to be a

Yield-Point ($Y.P.$), slightly above or beyond the ELASTIC LIMIT, stress at which the specimen begins to stretch without increase in This stress may be determined from a test without the use of deli-ing-apparatus by the DROP OF THE BEAM or HALT IN THE GAUGE ng-machine.

Ultimate Strength ($U.S.$) is the greatest unit stress the specimen

Rupture-Stress (R) is the unit stress at the time of failure. *This is* ess at the point of failure af*ter the area of the cross-section of the* s been *reduced; and because of the rapid dropping off of the load* *to determine. It is not regularly observed in testing,* attention being called to it merely to emphasize the fact that the ULTIMATE STRE steel is not the stress at the time of failure of the specimen. This is tru for wrought iron and ductile materials in general.



The horizontal scale for the distance $a\ b$ is ten times greater than for the remaining distance

Fig. 1. Stress-strain Diagram of Test on Steel Specimens.

Effect of Punching and Shearing. Structural steel is hardened action of the punch and shear in the process of manufacture in the sh the die-side the metal is forced to flow from the tool and this cold hardens and injures it as may be shown by a cold-bend test. The eff be removed by annealing; but in the best work it is usually specif rivet-holes shall be reamed during the assembling of the parts. This the injured metal and brings the parts into better alinement for the i of the rivets. The injury from shearing may be removed by milling the edges.

The Coefficient of Expansion of steel is 0.000 006 5 per degree Fah The ELONGATION in a length l, due to a change in temperature of t de then

$$e = 0.000\ 006\ 5\ lt$$

in which l is expressed in inches and t in degrees Fahrenheit.

The Weight of Steel is taken at 489.6 lb per cu ft. The sectional a member in square inches multiplied by 3.4 equals the weight in pou linear foot.

The Working Stress for structural steel in tension in buildings and b 16 000 lb per sq in in most specifications and building laws. For memb ject to constant load some designers use a WORKING STRESS of 20 000 lb

ıdard Specifications * for Structural Steel for Buildings

ions. Structural steel may bepu rchased under the specifications
iation of American Steel Manufacturers or of the American Society
Materials. Extracts from these specifications follow:

ıre. (1) Structural steel may be made by either the open-hearth
process.
l and plate or angle-material over ¾ in thick, which is to be punched,
le by the open-hearth process.

and Physical Properties. (2) They shall conform to these limits:

Properties considered	Structural steel	Rivet steel, open hearth
., maximum, Bessemer..........	0.10 per cent	....
., maximum, open-hearth........	0.06 per cent	0.06 per cent
naximum........................	....	0.045 per cent
ensile strength, lb per sq in.......	55 000-65 000	46 000-56 000
....	½ ult. tens. str.	½ ult. tens. str.
. min per cent in 8 in (Fig. 2)......	1 400 000* / Ult. tens. str.	1 400 000 / Ult. tens. str.
without fracture................	180° to diameter of 1 thickness	180° flat

* See Percentage of Elongation, page 384.

urposes of these specifications, the yield-point shall be determined by
observation of the drop of the beam of the testing-machine.

Determinations. (3) In order to determine if the material conforms
ical limitations prescribed in paragraph (2) herein, analysis shall be
e manufacturer from a test-ingot taken at the time of the pouring
lt or blow
d a correct
ch analysis
urnished to
r or his in-

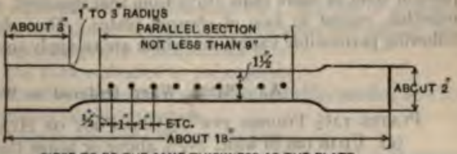


Fig. 2. Form of Specimen for Steel-test

Specimens.
ns for ten-
nd bending-
be made by
n the finished product coupons, which shall have both faces rolled
dges milled to the form shown in Fig. 2; or both edges parallel;
r be turned to a diameter of ¾ in for a length of at least 9 in and have
ds.
naterial more than ¾ in thick the bending-test specimen may be
in section.
t-rounds and small rolled bars shall be tested as rolled.

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s of form or substance of matter *of some paragraphs, as (3), (4), (5), (7)*
re been *proposed or adopted (1914-15). For amendments see Year Books*
c. for Test. Mats.

the thickest and thinnest material rolled. Should either of these test develop flaws, or should the tension-test specimen break outside of t third of its gauged length, it may be discarded and another tes substituted therefor. In case a tension-test specimen does not meet th tion, additional tests may be made.

(c) The bending-test may be made by pressure or by blows.

Modifications in Elongation for Thin and Thick Material. (7) F less than $\frac{5}{16}$ in and more than $\frac{3}{4}$ in in thickness, the following m shall be made in the requirements for elongation:

(d) For each increase of $\frac{1}{8}$ in in thickness above $\frac{3}{4}$ in, a deductio be made from the specified percentage of elongation.

(e) For each decrease of $\frac{1}{16}$ in in thickness below $\frac{5}{16}$ in, a deduc shall be made from the specified percentage of elongation.

(f) For pins, the required percentage of elongation shall be 5 less specified in the tabulation of paragraph (2), as determined on a test the center of which shall be 1 in from the surface.

Finish. (8) Finished material must be free from injurious seam cracks, and have a workmanlike finish.

Branding. (9) Test-specimens and every finished piece of ste stamped with melt or blow-number, except that small pieces may in bundles securely wired together, with the melt or blow-number tag attached.

Variation in Weight. (10) A variation in cross-section or weig piece of steel, of more than $2\frac{1}{2}\%$ from that specified, will be sufficien rejection, except in case of sheared plates. These latter are cove following permissible variations, which are to apply to single plates:

(A) Plates, When Ordered to Weight

PLATES $12\frac{1}{2}$ POUNDS PER SQUARE FOOT OR HEAVIER:

(g) Up to 100 in wide, $2\frac{1}{2}\%$ above or below the specified wei

(h) 100 in wide and over, 5% above or below.

PLATES ¼ INCH AND OVER IN THICKNESS

ess d,	Nominal weights, lb per sq ft	Width of plate			
		Up to 75 in %	75 in and up to 100 in %	100 in and up to 115 in %	Over 115 in %
	10.20	10	14	18	....
	12.75	8	12	16	....
	15.30	7	10	13	17
	17.85	6	8	10	13
	20.40	5	7	9	12
	22.95	4½	6½	8½	11
	25.50	4	6	8	10
	.....	3½	5	6½	9

PLATES UNDER ¼ INCH IN THICKNESS

ordered,	Nominal weights, lb per sq ft	Width of plate		
		Up to 50 in %	50 in and up to 70 in %	Over 70 in %
to 5/32	5.10 to 6.37	10	15	20
to 3/16	6.37 to 7.65	8½	12½	17
to ¼	7.65 to 10.20	7	10	15

on. (11) The inspector representing the purchaser shall have all facilities afforded to him by the manufacturer to satisfy him that the aterial is furnished in accordance with these specifications. All tests tions shall be made at the place of manufacture, prior to shipment.

al Paragraphs. The general specifications for a building of ordinary on should include in addition to the above, governing the quality and e material, paragraphs giving specifications for:

exact scope of the work embodied in the contract.

vision for the inspection of material and workmanship, stating specifi- is to bear the expense of the inspection.

cial tests; for example, tests of full-size eye-bars.

, limiting pitch and allowable stress for shop-rivets and field-rivets.

ning of surfaces and shop-painting.

6. Tension-Members

The best section for tension-members of relatively small size depends the kind of end-connections used. Angles or channels are generally veted connections. For very small members rectangular bars, such as , may be used. The strength of such members is computed on the net gh the rivet-holes. Angles used in tension should have lugs riveted standing legs and the tie-plate for the better distribution of the stress section. Tests on angles with riveted connections reported by F. pen* gave from 77 to 86% of the strength of the material as shown by

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roceedings of the American Society for Testing Materials, Vol. VI.

thickness as the bar. The eye is accurately drilled in position in the a
bar, true to diameter and exact central distance. Because of its a
for forging, soft steel is used in making eye-bars. They are also car
nealed before drilling. Table VI gives the dimensions of STANDARD
manufactured by the mills of the American Bridge Company. T
are of practically the same dimensions as the standard bars of ot
panies. There is from 34 to 42% excess material in the section throu
to insure in the forged part the development of the full strength of th
the bar. Standard bars should be used in design to avoid the expense
special dies in which to form the heads. Bars of less than the given
thickness are liable to fail, when loaded, by buckling in the head. T
increase the BENDING-STRESSES in the pins and thus, indirectly, the
size of the eye. Except for very large structures they are limited
2 in.

Tests of Full-Size Eye-Bars are generally required when a great
them are to be used in a structure, one in every fifty bars being usua
The specifications for carbon-steel bars require that an ULTIMAT
STRENGTH of 56 000 lb per sq in shall be developed, that the ELONGAT
whole length shall be 10% and that failure shall occur in the body
Nickel steel has been used for tension-members on a few long-span bri
WORKING STRESS on the eye-bars was increased about one-half over th

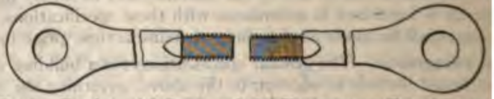


Fig. 3. Eye-bar with Screw-ends for Sleeve-nut or Turn-buckle

carbon steel, and the requirements of the test-bars made correspondin
The eye is made 1/50 in greater than the diameter of the pin. Bars
the same pins are drilled at the same setting so as to be of exactly
length. Bars must be true to length within 1/32 in. Small eye-bars

open turnbuckle has the of being easily inspected at the thread has sufficient nd that the ends of the t butt together.

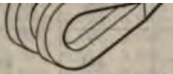


Fig. 5. Forked Loop

Screw-Ends are threaded enlargements on the ends of rods or bolts o give to the threaded portions a strength as great as that of the body . Because of effects of forging it is necessary to make the area of the on of the upset end at the root of the thread a little larger than that itself. A standard upset rod will fail in the body of the bar with- ging the threaded portion enough to prevent the turning of the nuts. nsions given are nearly the same with all manufacturers. If upset not be obtained the section-area at the root of the thread must be mputing the safe load.

s. Table IX, page 398, gives the dimensions and other details for cording to the latest standards of the American Bridge Company.

The following tables will be found useful in designing tension- or for drawing turnbuckles, sleeve-nuts, clevises, etc. The strength ds in Table II is based on the area at the root of the thread. For d weights of tie-rods and anchors for steel beams, see Table XIX, V.

Table II. Safe Loads in Pounds on Round Rods

Diameter inches	Plain rods Load in pounds based on area at root of thread			Upset rods Load in pounds based o area of rod	
	Stress in lb per sq in			Stress in lb per sq i	
	10 000	12 000	16 000	10 000	12 000
¼	270	324	432	491	590
5/16	450	540	720	767	920
⅜	680	816	1 088	1 104	1 320
7/16	930	1 116	1 488	1 503	1 800
½	1 260	1 513	2 016	1 963	2 360
9/16	1 620	1 944	2 592	2 485	2 970
⅝	2 020	2 424	3 232	3 068	3 680
¾	3 020	3 624	4 832	4 418	5 300
⅞	4 200	5 040	6 720	6 013	7 210
1	5 500	6 600	8 800	7 854	9 420
1⅛	6 940	8 328	11 104	9 940	11 930
1¼	8 930	10 716	14 288	12 270	14 720
1⅜	10 570	12 680	16 910	14 840	17 810
1½	12 950	15 540	20 720	17 670	21 200
1⅝	15 150	18 180	24 240	20 730	24 880
1¾	17 440	20 930	27 900	24 050	28 860
1⅞	20 480	24 580	32 760	27 610	33 130
2	23 020	27 620	36 830	31 420	37 700
2⅛	26 340	31 610	42 150	35 460	42 550
2¼	30 230	36 280	48 370	39 760	47 710
2⅜	33 000	39 600	52 800	44 300	53 160
2½	37 150	44 630	59 440	49 080	58 900
2¾	46 190	55 430	73 900	59 390	71 270
3	54 280	65 140	86 850	70 680	84 820
3¼	65 100	78 120	104 160	82 950	99 540
3½	75 480	90 570	120 770	96 210	115 450
3¾	86 410	103 690	138 250	110 450	132 540
4	99 930	119 920	159 890	125 660	150 790
4¼	113 290	135 900	181 300	141 800	170 160
4½	127 430	152 900	203 900	159 000	190 800
4¾	142 200	170 600	227 500	177 200	212 640
5	157 630	189 100	252 200	196 300	235 560
5¼	175 720	210 800	281 100	216 400	259 680
5½	192 670	231 200	308 300	237 500	285 000
5¾	212 620	255 100	340 200	259 600	311 000
6	230 980	277 200	369 600	282 700	339 200

1 250	1 500	1 750	2 000	2 250	2 500	2 750	3 000	3 250
2 500	3 000	3 500	4 000	4 500	5 000	5 500	6 000	6 500
3 750	4 500	5 250	6 000	6 750	7 500	8 250	9 000	9 750
5 000	6 000	7 000	8 000	9 000	10 000	11 000	12 000	13 000
6 250	7 500	8 750	10 000	11 250	12 500	13 750	15 000	16 250
7 500	9 000	10 500	12 000	13 500	15 000	16 500	18 000	19 500
8 750	10 500	12 250	14 000	15 750	17 500	19 250	21 000	22 750
10 000	12 000	14 000	16 000	18 000	20 000	22 000	24 000	26 000
11 250	13 500	15 750	18 000	20 250	22 500	24 750	27 000	29 250
12 500	15 000	17 500	20 000	22 500	25 000	27 500	30 000	32 500
13 750	16 500	19 250	22 000	24 750	27 500	30 250	33 000	36 750
15 000	18 000	21 000	24 000	27 000	30 000	33 000	36 000	39 000
16 250	19 500	22 750	26 000	29 250	32 500	35 750	39 000	42 250
17 500	21 000	24 500	28 000	31 500	35 000	38 500	42 000	45 500
18 750	22 500	26 250	30 000	33 750	37 500	41 250	45 000	48 750
20 000	24 000	28 000	32 000	36 000	40 000	44 000	48 000	52 000
21 250	25 500	29 750	34 000	38 250	42 500	46 750	51 000	55 250
22 500	27 000	31 500	36 000	40 500	45 000	49 500	54 000	58 500
23 750	28 500	33 250	38 000	42 750	47 500	52 250	57 000	61 750
25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000	65 000
27 500	33 000	38 500	44 000	49 500	55 000	60 500	66 000	71 500
30 000	36 000	42 000	48 000	54 000	60 000	66 000	72 000	78 000
32 500	39 000	45 500	52 000	58 500	65 000	71 500	78 000	84 500
35 000	42 000	49 000	56 000	63 000	70 000	77 000	84 000	91 000
37 500	45 000	53 500	60 000	67 500	75 000	83 500	90 000	97 500
40 000	48 000	56 000	64 000	72 000	80 000	88 000	96 000	104 000

Table III (Continued). Safe Loads in Pounds for Flat Rolled B

Computed for a stress of 16 000 pounds per square inch

Thickness in inches	Width in inches								
	3½	3¾	4	4¼	4½	4¾	5	5½	6
1/16	3 500	3 750	4 000	4 250	4 500	4 750	5 000	5 500	6 00
⅛	7 000	7 500	8 000	8 500	9 000	9 500	10 000	11 000	12 00
3/16	10 500	11 250	12 000	12 750	13 500	14 250	15 000	16 500	18 00
¼	14 000	15 000	16 000	17 000	18 000	19 000	20 000	22 000	24 00
5/16	17 500	18 750	20 000	21 250	22 500	23 750	25 000	27 500	30 00
⅜	21 000	22 500	24 000	25 500	27 000	28 500	30 000	33 000	36 00
7/16	24 500	26 250	28 000	29 750	31 500	33 250	35 000	38 500	42 00
½	28 000	30 000	32 000	34 000	36 000	38 000	40 000	44 000	48 00
9/16	31 500	33 750	36 000	38 250	40 500	42 750	45 000	49 500	54 00
⅝	35 000	37 500	40 000	42 500	45 000	47 500	50 000	55 000	60 00
11/16	38 500	41 250	44 000	46 750	49 500	52 250	55 000	60 500	66 00
¾	42 000	45 000	48 000	51 000	54 000	57 000	60 000	66 000	72 00
13/16	45 500	48 750	52 000	55 250	58 500	61 750	65 000	71 500	78 00
⅞	49 000	52 500	56 000	59 500	63 000	66 500	70 000	77 000	84 00
15/16	52 500	56 250	60 000	63 750	67 500	71 250	75 000	82 500	90 00
1	56 000	60 000	64 000	68 000	72 000	76 000	80 000	88 000	96 00
1 1/16	59 500	63 750	68 000	72 250	76 500	80 750	85 000	93 500	102 00
1⅛	63 000	67 500	72 000	76 500	81 000	85 500	90 000	99 000	108 00
1 3/16	66 500	71 250	76 000	80 750	85 500	90 250	95 000	104 500	114 00
1¼	70 000	75 000	80 000	85 000	90 000	95 000	100 000	110 000	120 00
1⅜	77 000	82 500	88 000	93 500	99 000	104 500	110 000	121 000	132 0
1½	84 000	90 000	96 000	102 000	108 000	114 000	120 000	132 000	144 0
1⅝	91 000	97 500	104 000	110 500	117 000	123 500	130 000	143 000	156 0
1¾	98 000	105 000	112 000	119 000	126 000	133 000	140 000	154 000	168 0
1⅞	105 000	112 500	120 000	127 500	135 000	142 500	150 000	165 000	180
2	112 000	120 000	128 000	136 000	144 000	152 000	160 000	176 000	192

630	780	940	1 090	1 250	1 410	1 560	1 720	1 880	2 030
1 250	1 560	1 880	2 190	2 500	2 810	3 130	3 440	3 750	4 060
1 880	2 340	2 810	3 280	3 750	4 220	4 690	5 160	5 630	6 090
2 500	3 130	3 750	4 380	5 000	5 630	6 250	6 880	7 500	8 130
3 130	3 910	4 690	5 470	6 250	7 030	7 810	8 590	9 380	10 200
3 750	4 690	5 630	6 560	7 500	8 440	9 380	10 300	11 300	12 200
4 380	5 470	6 560	7 660	8 750	9 840	10 900	12 000	13 100	14 200
5 000	6 250	7 500	8 750	10 000	11 300	12 500	13 800	15 000	16 300
5 630	7 030	8 440	9 840	11 300	12 700	14 100	15 500	16 900	18 300
6 250	7 810	9 380	10 900	12 500	14 100	15 600	17 200	18 800	20 300
6 880	8 590	10 300	12 000	13 800	15 500	17 200	18 900	20 600	22 300
7 500	9 380	11 300	13 100	15 000	16 900	18 800	20 600	22 500	24 400
8 130	10 200	12 200	14 200	16 300	18 300	20 300	22 300	24 400	26 400
8 750	10 900	13 100	15 300	17 500	19 700	21 900	24 100	26 300	28 400
9 380	11 700	14 100	16 400	18 800	21 100	23 400	25 800	28 100	30 500
10 000	12 500	15 000	17 500	20 000	22 500	25 000	27 500	30 000	32 500
10 600	13 300	15 900	18 600	21 300	23 900	26 600	29 200	31 900	34 500
11 300	14 100	16 900	19 700	22 500	25 300	28 100	30 900	33 800	36 600
11 900	14 800	17 800	20 800	23 800	26 700	29 700	32 700	35 600	38 600
12 500	15 600	18 800	21 900	25 000	28 100	31 300	34 400	37 500	40 600
13 800	17 200	20 600	24 100	27 500	30 900	34 400	37 800	41 300	44 700
15 000	18 800	22 500	26 300	30 000	33 800	37 500	41 300	45 000	48 800
16 300	20 300	24 400	28 400	32 500	36 600	40 600	44 700	48 800	52 800
17 500	21 900	26 300	30 600	35 000	39 400	43 800	48 100	52 500	56 900
18 800	23 400	28 100	32 800	37 500	42 200	46 900	51 600	56 300	60 900
20 000	25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000	65 000

stresses of 12 000, 12 500, and 15 000 lb increase by ⅕, ¼, and ½ respec-

g strength of wrought iron and steel, see pages 376 and 382.

Table IV (Continued). Safe Loads in Pounds for Flat Rolled B

Computed for a stress of 10 000 lb per square inch

Thickness in inches	Width in inches								
	3½	3¾	4	4¼	4½	4¾	5	5½	6
¹⁄₁₆	2 190	2 340	2 500	2 660	2 810	2 970	3 130	3 440	3 750
⅛	4 380	4 690	5 000	5 310	5 630	5 940	6 250	6 880	7 500
³⁄₁₆	6 560	7 030	7 500	7 970	8 440	8 910	9 380	10 300	11 300
¼	8 750	9 380	10 000	10 600	11 300	11 900	12 500	13 800	15 000
⁵⁄₁₆	10 900	11 700	12 500	13 300	14 100	14 800	15 600	17 200	18 800
⅜	13 100	14 100	15 000	15 900	16 900	17 800	18 800	20 600	22 500
⁷⁄₁₆	15 300	16 400	17 500	18 600	19 700	20 800	21 900	24 100	26 300
½	17 500	18 800	20 000	21 300	22 500	23 800	25 000	27 500	30 000
⁹⁄₁₆	19 700	21 100	22 500	23 900	25 300	26 700	28 100	30 900	33 800
⅝	21 900	23 400	25 000	26 600	28 100	29 700	31 300	34 400	37 500
¹¹⁄₁₆	24 100	25 800	27 500	29 200	30 900	32 700	34 400	37 800	41 300
¾	26 300	28 100	30 000	31 900	33 800	35 600	37 500	41 300	45 000
¹³⁄₁₆	28 400	30 500	32 500	34 500	36 600	38 600	40 600	44 700	48 800
⅞	30 600	32 800	35 000	37 200	39 400	41 600	43 800	48 100	52 500
¹⁵⁄₁₆	32 800	35 200	37 500	39 800	42 200	44 500	46 900	51 600	56 300
1	35 000	37 500	40 000	42 500	45 000	47 500	50 000	55 000	60 000
1¹⁄₁₆	37 200	39 800	42 500	45 200	47 800	50 500	53 100	58 400	63 800
1⅛	39 400	42 200	45 000	47 800	50 600	53 400	56 300	61 900	67 500
1³⁄₁₆	41 600	44 500	47 500	50 500	53 400	56 400	59 400	65 300	71 300
1¼	43 800	46 900	50 000	53 100	56 300	59 400	62 500	68 800	75 000
1⅜	48 100	51 600	55 000	58 400	61 900	65 300	68 800	75 600	82 500
1½	52 500	56 300	60 000	63 800	67 500	71 300	75 000	82 500	90 000
1⅝	56 900	60 900	65 000	69 100	73 100	77 200	81 300	89 400	97 500
1¾	61 300	65 600	70 000	74 400	78 800	83 100	87 500	96 300	105 000
1⅞	65 600	70 300	75 000	79 700	84 400	89 100	93 800	103 100	112 500
2	70 000	75 000	80 000	85 000	90 000	95 000	100 000	110 000	120 000

* See foot-note, preceding table.

Standard Proportions of Upset Screw-Ends for Round and Square Bars

Round bars			Square bars			
Diam. of screw at root of thread in	Number of threads per inch	Excess of effective area of screw-end over bar %	Diam. of upset screw-end in	Diam. of screw at root of thread in	Number of threads per inch	Excess of effective area of screw-end over bar %
0.620	10	54	¾	0.620	10	21
0.620	10	21	⅞	0.731	9	33
0.731	9	37	1	0.837	8	41
0.837	8	48	1	0.837	8	17
0.837	8	25	1⅛	0.940	7	23
0.940	7	34	1¼	1.065	7	35
1.065	7	48	1⅜	1.160	6	38
1.065	7	29	1⅜	1.160	6	20
1.160	6	35	1½	1.284	6	29
1.160	6	19	1⅝	1.389	5½	34
1.284	6	30	1⅝	1.389	5½	20
1.284	6	17	1¾	1.490	5	24
1.389	5½	23	1⅞	1.615	5	31
1.490	5	29	1⅞	1.615	5	19
1.490	5	18	2	1.712	4½	22
1.615	5	26	2⅛	1.837	4½	28
1.712	4½	30	2⅛	1.837	4½	18
1.712	4½	20	2¼	1.962	4½	24
1.837	4½	28	2⅜	2.087	4½	30
1.837	4½	18	2⅜	2.087	4½	20
1.962	4½	26	2½	2.175	4	21
1.962	4½	17	2⅝	2.300	4	26
2.087	4½	24	2⅝	2.300	4	18
2.175	4	26	2¾	2.425	4	23
2.175	4	18	2⅞	2.550	4	28
2.300	4	24	2⅞	2.550	4	20
2.300	4	17	3	2.629	3½	20
2.425	4	23	3⅛	2.754	3½	24

Table V (Continued). Standard Proportions of Upset Screw-Ends
Round and Square Bars

Diam. of round or side of square bar in	Round bars				Square bars		
	Diam. of upset screw-end in	Diam. of screw at root of thread in	Number of threads per inch	Excess of effective area of screw-end over bar %	Diam. of upset screw-end in	Diam. of screw at root of thread in	Number of threads per inch
2¼	2⅞	2.550	4	28	3⅛	2.754	3½
2⁵⁄₁₆	2⅞	2.550	4	22	3¼	2.879	3½
2⅜	3	2.629	3½	23	3⅜	3.004	3½
2⁷⁄₁₆	3⅛	2.754	3½	28	3⅜	3.004	3½
2½	3⅛	2.754	3½	21	3½	3.100	3¼
2⁹⁄₁₆	3¼	2.879	3½	26	3⅝	3.225	3¼
2⅝	3¼	2.879	3½	20	3⅝	3.225	3¼
2¹¹⁄₁₆	3⅜	3.004	3½	25	3¾	3.317	3
2¾	3⅜	3.004	3½	19	3⅞	3.442	3
2¹³⁄₁₆	3½	3.100	3¼	22	3⅞	3.442	3
2⅞	3⅝	3.225	3¼	26	4	3.567	3
2¹⁵⁄₁₆	3⅝	3.225	3¼	21	4⅛	3.692	3
3	3¾	3.317	3	22	4⅛	3.692	3
3⅛	3⅞	3.442	3	21	4⅜	3.923	2⅞
3¼	4	3.567	3	20	4½	4.028	2¾
3⅜	4⅛	3.692	3	20	4⅝	4.153	2¾
3½	4¼	3.798	2⅞	18	...	.....	...
3⅝	4½	4.028	2¾	23	...	.....	...
3¾	4⅝	4.153	2¾	23	...	.....	...
3⅞	4¾	4.255	2⅝	21	...	.....	...

REMARKS. As upsetting reduces the strength of iron, bars having the same
at the root of the thread as that of the bar invariably break in the screw-
tested to destruction, without developing the full strength of the bar. It is
necessary to make up for this loss in strength by an excess of metal in the up
ends over that in the bar.

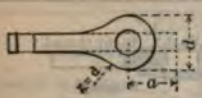
Table V is the result of numerous tests on finished bars made at the Keysto
Company's Works in Pittsburgh, Pa., and gives proportions that will cause
break in the body rather than in the upset end.

The screw-threads in the above table are the Franklin Institute standards.

To make one upset end for a 5-in length of thread, allow 6 in in length of
tional.

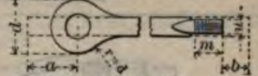
Table VI.* **Steel Eye-Bars** (AMERICAN BRIDGE COMPANY STANDARD)

ORDINARY EYE-BAR



Bar			Head				
	Thickness		Dia. d, in	Maximum pin		Additional material, a, ft and in	
	Max, in	Min, in		Dia. in	Excess head over bar, %	For ordering bar	For figuring w't
2	1	½	4½ 5½ † 6½	1¾ 2¾ 3¾	37.5	1- 0 1- 4 1- 9	0- 7 0-11 1- 4
2½	1	⅝	6 7 † 8	2½ 3½ 4½	40.0	1- 3 1- 7 2- 0	0-10 1- 2 1- 7
3	1½	⅝	7½ 8½ † 9½	3¼ 4¼ 5¼	41.7	1- 6 1-11 2- 4	1- 1 1- 5 1-10
4	1¾	¾ ⅞ 1	10 11 †12	4½ 5½ 6½	37.5	1-11 2- 3 2- 8	1- 6 1-10 2- 2
5	2	¾ 1 1	12 13½ †15	5¼ 6¾ 8¼	35.0	2- 1 2- 8 3- 3	1- 8 2- 2 2- 9
6	2	¾ 1 1	14 14¾ †16½	5¾ 6½ 8¼	37.5	2- 4 2- 6 3- 2	1-10 2- 1 2- 8
7	2	1 1⅛ 1⅛	16½ 17½ †18½	7 8 9	35.7	2- 7 2-11 3- 4	2- 2 2- 6 2-11
8	2	1 1⅛ 1¼	18 19 †20	7 8 9	37.5	2- 8 3- 0 3- 4	2- 3 2- 6 2-11
9	2	1⅛ 1¼	20 22	7½ 9½	38.9	2-11 3- 7	2- 6 3- 1
10	2	1⅛ 1¼ 1⅜	22½ 24 †25	9 10½ 11½	35.0	3- 5 3- 9 4- 1	2-10 3- 3 3- 7
2	2	1¼ 1⅜ 1½	26½ 28 †29½	10 11½ 13	37.5	3- 8 4- 2 4- 8	3- 3 3- 8 4- 1
4	2	1⅜ 1½ 1⅝	31 33 †34	12 14 15	35.7	4- 3 4-10 *5- 5*	3- 9 4- 4 *4- 8*
	2	*1¾* *1⅞*	*36* *†37½*	*14* *16*	*37.5* *34.4*	*4-11* *5- 5*	*4- 5* *4-10*

ADJUSTABLE EYE-BAR



Minimum length of short end from center of pin to end of screw, 6 ft, preferably 7 ft.

Thread on short end to be left hand

Pitch and shape of thread A. B. Co standard

Bar		Screw-end				
Width, in	Min thickness, in	Dia. u, in	Excess upset over bar, %	L'th m, in	Additional material, b, ft and in	
					For ordering bar	For figuring w't
2	† ⅝ ¾ ⅞	1¾ 1⅞ 2	39.6 36.6 31.4	4 4½ 4½	1- 0 1- 0 0-11	8 7½ 7½
2½	† ¾ ⅞ 1	2⅛ 2¼ 2⅜	41.2 38.1 36.7	4½ 5 5	1- 0 1- 0 1- 0	8 8 7½
3	† ¾ ⅞ 1	2¼ 2½ 2½	34.3 41.6 23.9	5 5½ 5½	1- 0 1- 1 1- 1	7½ 9½ 8½
4	† ¾ ⅞ 1 1⅛	2½ 2¾ 3 3¼	23.9 32.0 35.7 44.6	5½ 5½ 6 6½	1- 1 0-11 1- 1 1- 2	8½ 7½ 8½ 9½
5	† ¾ ⅞ 1 1⅛ 1¼	2⅞ 3 3¼ 3½ 3¾	36.2 24.1 30.2 34.2 38.3	6 6 6½ 7 7	1- 0 0-11 1- 0 1- 1 1- 2	8 7 8 8½ 9
6	†1 1⅛ 1¼ 1⅜	3½ 3¾ 4 4¼	25.8 28.0 33.2 37.3	7 7 7½ 8	1- 0 1- 0 1- 1 1- 2	7½ 8 8½ 9½
7	†1⅛ 1¼ 1⅜ 1½	4 4¼ 4½ 4¾	26.9 29.5 32.4 35.4	7½ 8 8½ 8½	1- 0 1- 1 1- 2 1- 2	8 8½ 9 9½
8	†1⅛ 1¼ 1⅜ 1½ 1⅝	4¼ 4½ 4¾ 5 5¼	25.9 27.4 29.3 31.4 35.2	8 8½ 8½ 9 9½	1- 0 1- 1 1- 1 1- 2 1- 3	8 8½ 8½ 9 10

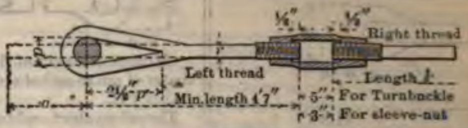
Bars marked † should be used only when absolutely unavoidable

Deduct pin-hole when figuring weight

From Pocket Companion, 1915 Edition, Carnegie Steel Company Pitt

Table VII.* Loop-Rods

AMERICAN BRIDGE COMPANY STANDARD



Pitch and shape of thread A. B. Co standard

Additional length A, in feet and inches, for one loop. $A=4.17p+5$

Diam. of pin, p, in	Diameter or side r of rod in inches									
	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1
1⅛	0- 9½	0-10	0-11	0-11½	....	....	....	....	....	...
1¼	0-10	0-10½	0-11½	1- 0	1- 1	....	....	....	....	...
1½	0-11	0-11½	1- 0½	1- 1	1- 2	1- 2½	....	....	....	...
1¾	1- 0	1- 0½	1- 1½	1- 2	1- 3	1- 3½	1- 4½	1- 5	1- 6	...
2	1- 1	1- 1½	1- 2½	1- 3	1- 4	1- 4½	1- 5½	1- 6	1- 7	1- 7
2¼	1- 2	1- 3	1- 3½	1- 4½	1- 5	1- 5½	1- 6½	1- 7	1- 8	1- 8
2½	1- 3	1- 4	1- 4½	1- 5½	1- 6	1- 7	1- 7½	1- 9	1- 9	1- 9
2¾	1- 4	1- 5	1- 5½	1- 6½	1- 7	1- 8	1- 8½	1- 9½	1-10	1-11
3	1- 5	1- 6	1- 6½	1- 7½	1- 8	1- 9	1- 9½	1-10½	1-11	2- 0
†3¼	1- 6	1- 7	1- 7½	1- 8½	1- 9	1-10	1-10½	1-11½	2- 0	2- 1
3½	1- 7½	1- 8	1- 8½	1- 9½	1-10	1-11	1-11½	2- 0½	2- 1	2- 2
†3¾	1- 8½	1- 9	1-10	1-10½	1-11	2- 0	2- 0½	2- 1½	2- 2	2- 3
4	1- 9½	1-10	1-11	1-11½	2- 0½	2- 1	2- 2	2- 2½	2- 3	2- 4
†4¼	....	1-11	2- 0	2- 0½	2- 1½	2- 2	2- 3	2- 3½	2- 4½	2- 5
4½	....	2- 0	2- 1	2- 1½	2- 2½	2- 3	2- 4	2- 4½	2- 5½	2- 6
†4¾	....	2- 1	2- 2	2- 2½	2- 3½	2- 4	2- 5	2- 5½	2- 6½	2- 7
5	....	2- 2½	2- 3	2- 3½	2- 4½	2- 5	2- 6	2- 6½	2- 7½	2- 8
†5¼	....	....	2- 4	2- 5	2- 5½	2- 6	2- 7	2- 7½	2- 8½	2- 9
5½	....	....	2- 5	2- 6	2- 6½	2- 7½	2- 8	2- 9	2- 9½	2-10
†5¾	....	....	2- 6	2- 7	2- 7½	2- 8½	2- 9	2-10	2-10½	2-11
6	....	....	2- 7	2- 8	2- 8½	2- 9½	2-10	2-11	2-11½	3- 0
†6¼	....	....	....	2- 9	2- 9½	2 10½	2-11	3- 0	3- 0½	3- 1
6½	....	....	....	2-10	2-10½	2-11½	3- 0	3- 1	3- 1½	3- 2
†6¾	....	....	....	2-11	3- 0	3- 0½	3- 1	3- 2	3- 2½	3- 3
7	....	....	....	3- 0	3- 1	3- 1½	3- 2½	3- 3	3- 3½	3- 4

Pins marked † are special. Maximum shipping length of l = 35 ft

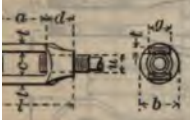
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitts

Table VIII.* Turnbuckles and Sleeve-Nuts

AMERICAN BRIDGE COMPANY STANDARD

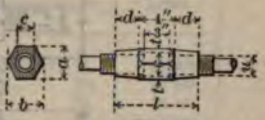
All dimensions in inches

TURNBUCKLES



turnbuckles marked†
of thread, A. B. Co standard

SLEEVE-NUTS



Pitch and shape of thread, A. B. Co standard

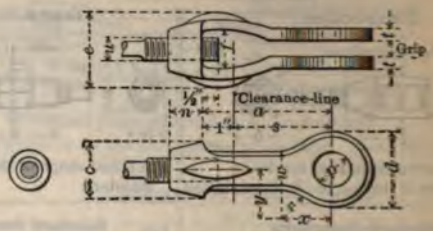
ndard dimensions						Standard dimensions						
c	t	g	b	W't, lb	Dia. of screw u	d	l	a	b	c	t	W't, lb
9/16	3/16	1/2	1 1/16	1	....	...	...	...	...	...	...	...
5/8	1/4	5/8	1 3/8	1	....	...	...	...	...	...	...	...
5/8	1/4	5/8	1 3/8	1	....	...	...	...	...	...	...	...
13/16	5/16	3/4	1 9/16	1 1/2	....	...	...	...	...	...	...	...
13/16	5/16	3/4	1 9/16	1 1/2	....	...	...	...	...	...	...	...
1 1/16	11/32	7/8	2	2	....	...	...	...	...	...	...	...
1 1/4	3/8	1	2 1/4	3	7/8	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3
1 5/16	7/16	1 1/4	2 7/16	4	1	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3
1 7/16	1/2	1 1/4	2 9/16	5	1 1/8	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	4
1 9/16	1/2	1 1/2	2 3/4	6	1 1/4	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	4
1 11/16	1/2	1 5/8	3 1/16	7	1 3/8	2	8	2 3/8	2 3/4	1 5/8	3/8	5
1 3/4	5/8	1 3/4	3 3/16	8	1 1/2	2	8	2 3/8	2 3/4	1 5/8	3/8	6
2	5/8	1 7/8	3 1/2	10	1 5/8	2 1/4	8 1/2	2 3/4	3 3/16	1 7/8	7/16	8
2 1/8	5/8	2	3 3/4	11	1 3/4	2 1/4	8 1/2	2 3/4	3 3/16	1 7/8	7/16	9
2 3/16	11/16	2 1/8	3 7/8	12	1 7/8	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	10
2 3/8	11/16	2 1/4	4 1/4	14	2	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	11
2 1/2	23/32	2 1/2	4 1/2	17	2 1/8	2 3/4	9 1/2	3 1/2	4 1/16	2 3/8	9/16	14
2 11/16	13/16	2 1/2	4 3/4	20	2 1/4	2 3/4	9 1/2	3 1/2	4 1/16	2 3/8	9/16	15
2 3/4	13/16	2 3/4	4 7/8	22	2 3/8	3	10	3 7/8	4 1/2	2 5/8	5/8	18
3 1/16	27/32	3	5 3/8	25	2 1/2	3	10	3 7/8	4 1/2	2 5/8	5/8	19
3 1/4	15/16	3 1/4	5 3/4	33	2 3/4	3 1/4	10 1/2	4 1/4	4 15/16	2 7/8	11/16	23
3 7/16	1 1/32	3 1/4	6 1/16	36	2 7/8	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	27
3 5/8	1 1/32	3 1/2	6 3/8	40	3	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	28
3 7/8	1 1/16	4	6 3/4	50	3 1/4	3 3/4	11 1/2	5	5 13/16	3 3/8	13/16	35
4 1/4	1 7/32	4	7 1/4	65	3 1/2	4	12	5 3/8	6 1/4	3 5/8	7/8	40
4 7/16	1 5/16	5	8 1/4	95	3 3/4	4 1/4	12 1/2	5 3/4	6 11/16	3 7/8	15/16	47
4 5/8	1 7/16	5	8 3/4	108	4	4 1/2	13	6 1/8	7 1/16	4 1/8	1	55
4 7/8	1 5/8	5 5/32	9 1/4	140	4 1/4	4 3/4	13 1/2	6 1/2	7 1/2	4 3/8	1 1/16	65
5 1/2	1 3/4	6 1/2	10 3/4	195	4 1/2	5	14	6 7/8	7 15/16	4 3/4	1 1/16	75
5 5/8	2	6 1/2	11 1/4	205	....	...	...	...	...	...	...	...
6	2 1/4	6 1/2	11 7/8	250	....	...	...	...	...	...	...	...

mpanion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table IX.* Clevises

AMERICAN BRIDGE COMPANY STANDARD

All dimensions in inches



Grip=thickness of plate+¼ in, but must not exceed dimension

Clevis-No.	Head								Nut					Fork	
	d	w	t	Max p	Min p	r	x	y	n	c	Max u	Min u	e	f	a
3	3	1½	½	1½	1	2¼	2¼	3	1½	2¼	1⅛	1	3 1/16	1¼	5
4	4	2	½	2	1¼	3	3	4	1¾	2⅞	1⅝	1⅛	3⅝	1¾	6
5	5	2½	⅝	2½	1½	3¾	3¾	5	2¼	3¾	2⅛	1½	4½	2¼	7
6	6	3	¾	3	2	4½	4½	6	2½	4⅜	2⅝	2	5⅜	2¾	8
7	7	3½	⅞	3½	2½	5¼	5¼	7	3	5	3	2¼	6 3/16	3¼	9

CLEVIS-NUMBERS FOR VARIOUS RODS AND PINS

Rods			Pins								
Round	Square	Upset	1	1¼	1½	1¾	2	2¼	2½	2¾	3
¾	.....	1	3	3	3	...	...	...	...	...	...
.....	¾	1⅛	3	3	3	4	4	...	...	...	...
⅞	⅞	1¼	...	4	4	4	4	...	...	...	...
1	.....	1⅜	...	4	4	4	4	...	...	...	...
1⅛	1	1½	...	4	4	4	4	5	5	...	...
1¼	1⅛	1⅝	...	4	4	4	4	5	5	...	...
1⅜	.....	1¾	...	...	5	5	5	5	5	...	...
.....	1¼	1⅞	...	...	5	5	5	5	5	...	...
1½	1⅜	2	...	...	5	5	5	5	5	6	6
1⅝	.....	2⅛	...	...	5	5	5	5	5	6	6
1¾	1½	2¼	...	...	...	...	6	6	6	6	6
1⅞	1⅝	2⅜	...	...	...	...	6	6	6	6	6
2	1¾	2½	...	...	...	...	6	6	6	6	6
2⅛	.....	2⅝	...	...	...	...	6	6	6	6	6
.....	1⅞	2¾	...	...	...	...	...	...	7	7	7
2¼	2	2⅞	...	...	...	...	...	...	7	7	7
2⅜	2⅛	3	...	...	...	...	...	...	7	7	7

Clevises above and to right of zigzag line may be used with forks strai
below and to left of this line should have forks closed so as not to overstress

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitt

le X. Safe Loads in Tension for Common Sizes of Angles with One ⅞-Inch Rivet-Hole for a ¾-Inch Rivet

Load in pounds for a stress of 16 000 lb per sq in

Size of angle	Load	Size of angle	Load
6×4×¾	100 500	3½×2½×⅝	45 000
⅝	85 000	9/16	41 100
½	68 900	½	37 000
		⅜	28 500
5×3½×¾	82 500	¼	19 500
⅝	70 100		
½	57 000	3×3×⅝	45 000
		½	37 000
5×3×¾	76 500	⅜	28 500
⅝	64 900	¼	19 500
½	53 000		
⅜	40 500	3×2½×½	33 000
		⅜	25 600
4×4×¾	76 500	5/16	21 800
⅜	40 500	¼	17 600
4×3½×⅝	60 000		
⅜	37 600	3×2×7/16*	25 900
		⅜*	25 600
4×3×⅝	55 000	5/16*	21 800
½	45 000	¼*	17 600
⅜	34 600		
		2½×2½×7/16	25 900
3½×3½×¾	64 500	⅜	22 600
⅝	55 000	5/16	19 200
½	45 000	¼	15 500
⅜	34 600		
		2½×2×7/16	22 400
3½×3×⅝	50 100	⅜	19 500
½	41 000	5/16	16 600
⅜	31 500	¼	12 800

se are special angles. It is better not to use them in ordinary work because of elay in delivery.

Thic of p	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16
3/16	0.05	0.06	0.07	0.08	0.09	0.11	0.12	0.13	0.14	0.15	0.16	0.18	0.19	0.20
1/4	0.06	0.08	0.09	0.11	0.13	0.14	0.16	0.17	0.19	0.20	0.22	0.23	0.25	0.27
5/16	0.08	0.10	0.12	0.14	0.16	0.18	0.20	0.21	0.23	0.25	0.27	0.29	0.31	0.33
3/8	0.09	0.12	0.14	0.16	0.19	0.21	0.23	0.26	0.28	0.30	0.33	0.35	0.38	0.40
7/16	0.11	0.14	0.16	0.19	0.22	0.25	0.27	0.30	0.33	0.36	0.38	0.41	0.44	0.46
1/2	0.13	0.16	0.19	0.22	0.25	0.28	0.31	0.34	0.38	0.41	0.44	0.47	0.50	0.53
9/16	0.14	0.18	0.21	0.25	0.28	0.32	0.35	0.39	0.42	0.46	0.49	0.53	0.56	0.60
5/8	0.16	0.20	0.23	0.27	0.31	0.35	0.39	0.43	0.47	0.51	0.55	0.59	0.63	0.66
11/16	0.17	0.21	0.26	0.30	0.34	0.39	0.43	0.47	0.52	0.56	0.60	0.64	0.69	0.73
3/4	0.19	0.23	0.28	0.33	0.38	0.42	0.47	0.52	0.56	0.61	0.66	0.70	0.75	0.80
13/16	0.20	0.25	0.30	0.36	0.41	0.46	0.51	0.56	0.61	0.66	0.71	0.76	0.81	0.86
7/8	0.22	0.27	0.33	0.38	0.44	0.49	0.55	0.60	0.66	0.71	0.77	0.82	0.88	0.93
15/16	0.23	0.29	0.35	0.41	0.47	0.53	0.59	0.64	0.70	0.76	0.82	0.88	0.94	1.00
1	0.25	0.31	0.38	0.44	0.50	0.56	0.63	0.69	0.75	0.81	0.88	0.94	1.00	1.06

* See also Table I, Chapter XX and paragraph, Punching Rivet-Holes,

7. Wire

Manufacture. Iron and steel wires are made from BILLETS about These are rolled into long rods which are dipped in acid to remo and furnish lubrication for the drawing process. This consists in rods while cold through steel dies having a series of holes of graduall diameters. The cold working of the metal hardens it and makes that it is necessary to anneal it at intervals during the process. increases the strength of the material, so that wires of different siz made of the same material, differ greatly in ULTIMATE STRENGTH.

Finish. The common grades of iron and steel wire are furnishe nce must be made to Table XII in connection with the foot-note to . The following table is arranged from values given in the Catalogue . Roebling's Sons Company:

XII. Approximate Ultimate Strength of Different Sizes of Iron and Steel Wire

Kind of wire	Ultimate strength	
	Large size lb per sq in	Small size lb per sq in
	45 000	60 000
and telephone (steel)	60 000	80 000
ator	247 000	285 000
	307 000	340 000
l wire	200 000	345 000
n copper trolley wire	50 000	not used
n telegraph and telephone copper	56 000	66 000

Table XIII. Comparison of Standard Gauges for Wire and She

Number of gauge	Diameter or thickness in decimals of an inch					
	Birmingham or Stubs iron-wire-gauge	American or Brown & Sharpe wire-gauge	United States standard gauge for sheet and plate iron and steel	Washburn & Moen Manufacturing Co.* and John A. Roebling's Sons Co. wire-gauge	Trenton Iron Co. wire-gauge	Ameri-can Screw Co. wire-gaug
0000000	......	....	0.5	0.4900	....	......
000000	......	0.580000	0.46875	0.4615	....	......
00000	0.500	0.516500	0.4375	0.4305	0.450	......
0000	0.454	0.460000	0.40625	0.3938	0.400	......
000	0.425	0.409642	0.375	0.3625	0.360	0.03
00	0.380	0.364796	0.34375	0.3310	0.330	0.04
0	0.340	0.324861	0.3125	0.3065	0.305	0.05
1	0.300	0.289297	0.28125	0.2830	0.285	0.071
2	0.284	0.257627	0.265625	0.2625	0.265	0.08
3	0.259	0.229423	0.25	0.2437	0.245	0.097
4	0.238	0.204307	0.234375	0.2253	0.225	0.11
5	0.220	0.181940	0.21875	0.2070	0.205	0.12
6	0.203	0.162023	0.203125	0.1920	0.190	0.13
7	0.180	0.144285	0.1875	0.1770	0.175	0.15
8	0.165	0.128490	0.171875	0.1620	0.160	0.16
9	0.148	0.114423	0.15625	0.1483	0.145	0.17
10	0.134	0.101897	0.140625	0.1350	0.130	0.18
11	0.120	0.090742	0.125	0.1205	0.1175	0.20
12	0.109	0.080808	0.109375	0.1055	0.105	0.21
13	0.095	0.071962	0.09375	0.0915	0.0925	0.22
14	0.083	0.064084	0.078125	0.0800	0.0806	0.24
15	0.072	0.057068	0.0703125	0.0720	0.070	0.25
16	0.065	0.050821	0.0625	0.0625	0.061	0.26
17	0.058	0.045257	0.05625	0.0540	0.0525	0.28
18	0.049	0.040303	0.05	0.0475	0.045	0.29
19	0.042	0.035890	0.04375	0.0410	0.040	0.30
20	0.035	0.031961	0.0375	0.0348	0.035	0.32
21	0.032	0.028462	0.034375	0.03175	0.031	0.33
22	0.028	0.025346	0.03125	0.0286	0.028	0.347
23	0.025	0.022572	0.028125	0.0258	0.025	0.36
24	0.022	0.020101	0.025	0.0230	0.0225	0.37
25	0.020	0.017900	0.021875	0.0204	0.020	0.38
26	0.018	0.015941	0.01875	0.0181	0.018	0.40
27	0.016	0.014195	0.0171875	0.0173	0.017	0.41
28	0.014	0.012641	0.015625	0.0162	0.016	0.4
29	0.013	0.011257	0.0140625	0.0150	0.015	0.43
30	0.012	0.010025	0.0125	0.0140	0.014	0.45
31	0.010	0.008928	0.0109375	0.0132	0.013	0.46
32	0.009	0.007950	0.01015625	0.0128	0.012	0.47
33	0.008	0.007080	0.009375	0.0118	0.011	0.49
34	0.007	0.006305	0.00859375	0.0104	0.010	0.50
35	0.005	0.005615	0.0078125	0.0095	0.0095	0.51
36	0.004	0.005000	0.00703125	0.0090	0.009	0.53
37	......	0.004453	0.006640625	0.0085	0.0085	0.54
38	......	0.003965	0.00625	0.0080	0.008	0.557
39	......	0.003531	....	0.0075	0.0075	0.57
40	......	0.003144	....	0.0070	0.007	0.58

---

* Now the American Steel and Wire Company.

The United States Standard Gauge was legalized by Act of Congress as a standard gauge for sheet and plate iron and steel, and is used by t Department and by sheet-plate and tin-plate manufacturers.

Table XIV. Weight, Length and Strength of Steel Wire

Gauge of J. A. Roebling's Sons Company

er, ing e	Diameter in	Area sq in	Breaking-load in pounds at rate of 100 000 lb per sq in	Weight in pounds		Number of feet in 2 000 pounds
				Per 1 000 ft	Per mile	
0	0.460	0.166191	16 619	558.4	2 948	3 582
0	0.430	0.145221	14 522	487.9	2 576	4 099
0	0.394	0.121304	12 130	407.6	2 152	4 907
0	0.362	0.102922	10 292	345.8	1 826	5 783
0	0.331	0.086049	8 605	289.1	1 527	6 917
0	0.307	0.074023	7 402	248.7	1 313	8 041
1	0.283	0.062902	6 290	211.4	1 116	9 463
2	0.263	0.054325	5 433	182.5	964	10 957
3	0.244	0.046760	4 676	157.1	830	12 730
4	0.225	0.039761	3 976	133.6	705	14 970
5	0.207	0.033654	3 365	113.1	597	17 687
6	0.192	0.028953	2 895	97.3	514	20 559
7	0.177	0.024606	2 461	82.7	437	24 191
8	0.162	0.020612	2 061	69.3	366	28 878
9	0.148	0.017203	1 720	57.8	305	34 600
0	0.135	0.014314	1 431	48.1	254	41 584
1	0.120	0.011310	1 131	38.0	201	52 631
2	0.105	0.008659	866	29.1	154	68 752
3	0.092	0.006648	665	22.3	118	89 525
4	0.080	0.005027	503	16.9	89.2	118 413
5	0.072	0.004071	407	13.7	72.2	146 198
6	0.063	0.003117	312	10.5	55.3	191 022
7	0.054	0.002290	229	7.70	40.6	259 909
8	0.047	0.001735	174	5.83	30.8	343 112
9	0.041	0.001320	132	4.44	23.4	450 856
0	0.035	0.000962	96	3.23	17.1	618 620

ble was calculated on a basis of 483.84 lb per cu ft for steel wire. Iron wire is hter.

eaking strengths were calculated for 100 000 lb per sq in throughout, simply nience, so that the breaking strengths per square inch of wires of any strength uickly determined by multiplying the values given in the table by the ratio he strength per square inch and 100 000. Thus, a No. 15 wire, with a strength e inch of 150 000 pounds, has a breaking strength of

$$407 \times \frac{150\ 000}{100\ 000} = 610.5 \text{ lb.}$$

t not be inferred from this table that steel wire invariably has a strength of per sq in. As a matter of fact its strength ranges from 45 000 lb per sq in for aled wire to over 400 000 lb per sq in for hard wire.

(1) **Haulage or Transmission-Rope,** composed of six strands o
each, laid about a hemp core. It is used for haulage, transmissior
places where surface-wear is of chief consideration and where sheav
diameter may be used.

(2) **Hoisting-Rope,** composed of six strands of nineteen wires
used for elevator service, shafts and derricks, and in places where
ject to abrasion and where flexibility is of chief consideration.

(3) **Seale Rope,** composed of six strands of nineteen wires ea
coils of the strands being of finer wire. It is intermediate in flexib
the first and second kinds of rope.

(4) **Non-Spinning Hoisting-Rope,** having eighteen strands of sev
Twelve of the strands are laid in reverse direction to the inne
it well adapted for hoisting in free suspension without untwisting
the load.

(5) **Extra-Flexible Hoisting-Rope,** having eight strands of ninete

(6) **Special Flexible Hoisting-Rope,** having six strands of thirt
each.

(7) **Hawser-Rope and Flexible Running-Rope,** having six stran
galvanized wires each, laid about a hemp core.

(8) **Tiller-Rope,** composed of six small seven-strand ropes laid a
core. It is the most flexible of wire ropes and is used to operate t
hand-ropes in elevators.

The Lay of Wire Rope is the twist of the wires in the strands
the strands in the rope. In the ORDINARY LAY the twist of the s
reverse of that of the wires, while in the LANG LAY the strands a
same direction as the twist of the wires. This latter gives a greate
of the wearing-surface and a somewhat greater flexibility; but it
advantage of a tendency to untwist and for this reason should no
hoisting weights in free suspension. Wire rope is also made up in
BON FORM. For large sizes it is more flexible than standard rope
run over smaller drums.

Table XV. Strength of Wire Rope

Arranged from the 1912 list of John A. Roebling's Sons Company

de ber	Diameter inches	Weight per foot, hemp core	Approximate breaking-load in pounds		Minimum diameter of drum or sheave in feet	
			Iron	Cast steel	Iron	Cast steel
			HOISTING-ROPE Six strands of nineteen wires each, about a hemp core			
	2¼	8.00	144 000	266 000	14	9
	2	6.30	110 000	212 000	12	8
½	1⅞	5.55	100 000	192 000	12	8
	1¾	4.85	88 000	170 000	11	7
	1⅝	4.15	76 000	144 000	10	6.5
	1½	3.55	66 000	128 000	9	6
½	1⅜	3	56 000	112 000	8.5	5.5
	1¼	2.45	45 600	94 000	7.5	5
	1⅛	2	37 200	76 000	7	4.5
	1	1.58	29 000	60 000	6	4
	⅞	1.20	23 600	46 000	5.5	3.5
	¾	0.89	17 000	35 000	4.5	3
	⅝	0.62	12 000	25 000	4	2.5
	9/16	0.50	9 400	20 000	3.5	2.25
	½	0.39	7 800	16 800	3	2
	7/16	0.30	5 800	13 000	2.75	1.75
	⅜	0.22	4 800	9 600	2.25	1.5
			STANDING ROPE Six strands of seven wires each			
	1½	3.55	64 000	126 000	16	11
	1⅜	3	56 000	106 000	15	10
	1¼	2.45	46 000	92 000	13	9
	1⅛	2	38 000	74 000	12	8
5	1	1.58	30 000	62 000	10.5	7
6	⅞	1.20	24 000	48 000	9	6
7	¾	0.89	17 600	37 200	7.5	5
18	11/16	0.75	14 600	30 800	7.25	4.75
19	⅝	0.62	12 000	26 000	7	4.50
20	9/16	0.50	9 600	20 000	6	4
21	½	0.39	7 400	15 400	5.5	3.5
22	7/16	0.30	5 200	11 000	4.5	3
23	⅜	0.22	4 400	9 200	4	2.75
24	5/16	0.15	3 400	7 000	3.5	2.25
25	9/32	0.125	2 400	5 000	3	1.75

e working load *is to be taken at one-fifth the breaking-load.* This is assumed in *lating the diameter of the sheaves.*

rapid corrosion of the rope. Because of the many kinds and qualities of
it is well to consult the manufacturers as to which kind will best suit the
tions for any particular service. The John A. Roebling's Sons Company,
ton, N. J., the Trenton Iron Company, Trenton, N. J., and A. Leschen &
Rope Company, St. Louis, Mo., are among the largest manufacturers of
lines of ropes.

Coils. Wire rope should not be coiled like hemp rope, and in order to
kinking, should be taken from the reels without twisting. If it is not
a reel, to avoid injury it must be rolled over the ground like a wheel.

Lubrication. It is very important that running ropes be properly
cated, since, if proper care is not taken, the wear on the interior parts,
the wires, may be almost as great as the outside abrasion. The oil should
trate to the core of the rope and yet not drip off a few days after appli
Information as to the care of rope may be obtained of the Wire Rope
ting Company, Newark, N. J.

Sheaves. The size of sheaves recommended in the tables are calculated
a working-load of one-fifth the given breaking-load. If smaller sheaves are
the life of the rope will be greatly shortened, because of the excessive be
of the outer wires.

Table XVI. Galvanized, Steel-Wire Strands

For guys, signal-cords, trolley-line span wire, etc. Taken from the Trenton Iron Company's full list

Diameter in inches	List-price per 100 feet	Weight per 100 feet	Approximate breaking-load in pounds
5/8	$7.25	80	14 000
9/16	5.75	65	11 000
1/2	4.50	52	8 500
7/16	3.75	41	6 500
3/8	2.75	30	5 000
5/16	2.25	22	3 800
1/4	1.75	13	2 300
7/32	1.50	9½	1 800
3/16	1.25	7½	1 400
5/32	1.15	5½	900
1/8	1.00	3¼	600
3/32	.80	2	400

9. Cotton, Hemp and Manila Rope

Rope is made of cotton, hemp, and Manila fiber. Cotton is used for
sizes, only, and for such purposes as sash-cord, etc.

Manufacture. In the manufacture of rope the fiber is first spun into
From twenty to eighty threads are twisted together into STRANDS and
strands, three or four, are laid together, opposite in direction to the
the strands, but in the same direction as the THREADS. This causes the
to be twisted as the rope untwists and produces a balancing of forces that
to keep the rope in shape.

Cables and Hawsers, are made up of strands of rope.

e used for Hoisting wears rapidly from the action of the pulleys and om the bending which causes a slight internal motion between the fibers hafing and grinding away of the interior.

edore-Rope, of the C. W. Hunt Company, is filled with a tallow and go lubricant which decreases the internal friction, lubricates the outside ope and thus greatly prolongs its life.

igth. The values of the strength of new rope, given in Table XVII, are om the Specifications of the United States Navy Department, issued , 1910, at the Boston Navy-Yard. Manufacturers generally adopt these d weights and claim a strength equal to or a little greater than the values The UNIT STRENGTH for the different sizes varies, being about 14 000 lb n for the smaller and about 10 000 for the largest size. The approx- rmula, offered by C. W. Hunt, of 720 times the square of the circum- in inches, is equivalent to about 9 000 lb per sq in. American hemp, as about 5% greater strength than Manila rope of the same size. The cifications give for two-strand American-hemp rope, 85% of the strength ree-strand rope of the same material.

Table XVII. Strength and Weight of Rope

Specifications of the United States Navy, June, 1910

	Diameters in	Manila hemp, plain-laid		American hemp, tarred, plain-laid, three strands	
		Weights lb per ft	Breaking-loads lb	Weights lb per ft	Breaking-loads lb
	0.24	0.02	700	0.051	750
	0.32	0.033	1 000	0.06	1 060
	0.40	0.05	1 800	0.067	1 670
	0.48	0.083	2 500	0.083	2 340
	0.56	0.10	3 000	0.105	3 325
	0.64	0.14	4 000	0.16	3 955
	0.72	0.17	5 000	0.21	4 720
	0.80	0.21	5 500	0.26	5 770
	0.87	0.26	6 600	0.32	7 000
	0.95	0.305	7 800	0.37	8 400
¼	1.03	0.36	9 200	0.44	9 800
½	1.16	0.42	10 500	0.51	11 200
¾	1.19	0.47	12 200	0.59	13 000
	1.27	0.54	13 700	0.67	14 550
½	1.43	0.67	17 400	....	......
5	1.59	0.83	21 800	....	......
5½	1.75	1.00	27 700	....	......
6	1.90	1.21	31 000	....	......
7	2.22	1.63	36 200	....	......
8	2.54	2.17	47 300	....	......
9	2.87	2.70	60 000	....	......
10	3.14	3.33	71 200	....	......

la-hemp rope is made in three strands and in sizes up to 3 in in circumference; ods are used for sizes larger than 3 in in circumference.

Table XVIII. Working Loads for Manila Rope

Working load = $C \times$ breaking-load of new rope

D = minimum diameter of sheave in inches

Speed	Feet per minute	Kind of work	Value of C
Slow......	50 to 100	Derrick, crane, quarry, etc.	0.014
Medium...	150 to 300	Wharf, cargo, etc.	0.056
Rapid.....	400 to 800	....	0.028

The wear in such service is very rapid, a $1\frac{1}{2}$-in rope wearing out i 7 000 to 10 000 tons of coal. On the other hand, a $1\frac{1}{2}$-in trans running at 5 000 ft per min and carrying 1 000 horse-power over and 17 ft in diameter, lasts for years, the difference being due tc stress and larger sheaves.

10. Chains

Manufacture. Chains are made by hand-welding from best bar. The bending and welding reduce the strength so that the twice but only from 1.55 to 1.70 times as strong as the original bar it was made. STUD-CHAIN having a bar welded across each link and prevent fouling in handling, is not as strong as OPEN-LINK CH Goodenough, in a Bulletin from the Illinois Engineering Experir finds that the stud-chain will support a greater load at the ELASTIC material. If P is the load, d the diameter of the bar and S the stre ulas are:

$$P = 0.5\, d^2 S \text{ for the stud-chain, and}$$

able XIX. Sizes, Weights, Proof-Tests and Average Breaking-Loads for Chains

Bradlee and Company, Philadelphia

Size of chains in	Approximate weight per foot	D.B.G. special crane		Crane	
		Proof-test lb	Average breaking-load lb	Proof-test lb	Average breaking-load lb
1/4	3/4	1 932	3 864	1 680	3 360
3/8	1 1/2	4 186	8 372	3 640	7 280
1/2	2.5	7 728	15 456	6 720	13 440
5/8	4.1	11 914	23 828	10 360	20 720
3/4	6.2	17 388	34 776	15 120	30 240
7/8	8.4	22 484	44 968	20 440	40 880
1	10.5	29 568	59 136	26 880	53 760
1 1/8	13.6	37 576	75 152	34 160	68 320
1 1/4	16	46 200	92 400	42 000	84 000
1 3/8	19.2	55 748	111 496	50 680	101 360
1 1/2	23	66 528	133 056	60 480	120 960
1 5/8	28	74 382	148 764		
1 3/4	31	82 320	164 640		
1 7/8	35	94 360	188 720		
2	40	107 520	215 040		
2 1/8	46.5	121 240	242 480		

The specifications of the United States Navy Department require the same proof-test is given above for crane-chain and a breaking-strength 10% greater than that given r special crane-chain.

Table XX. Proof-Tests and Average Breaking-Loads for Studded Chain-Cables

Specifications of the United States Navy Department

Size of cable in	Proof-test lb	Average breaking-load lb	Size of cable in	Proof-test lb	Average breaking-load lb
1	34 607	67 526	1 15/16	130 202	225 687
1 1/8	43 812	82 686	2	138 739	239 732
1 1/4	54 194	100 630	2 1/16	147 544	254 223
1 5/16	59 784	109 771	2 1/8	156 622	269 160
1 3/8	65 574	119 355	2 1/4	175 591	300 373
1 7/16	71 672	129 385	2 1/2	216 779	368 153
1 1/2	78 041	139 861	2 5/8	238 995	404 719
1 9/16	84 678	150 783	2 3/4	262 302	443 069
1 5/8	91 588	162 152	2 7/8	286 692	483 203
1 3/4	106 222	186 228	3	312 165	525 121
1 7/8	121 937	212 188	3 1/8	339 102	567 823

Strength of Old Iron. A square link 12 in broad, 1 in thick, and 12 ft long was taken from the Kieff Bridge, then forty years old, and comparison with a similar link which had been preserved in the stock- the bridge was built. The following is a record of a mean of four test-pieces, 1 by 1⅛ by 8 in, taken from each link.

Table XXI. Strength of Old Chain-Links*

	Old link from bridge	New link stock-
Tensile strength per sq in, tons	21.8	22
Elastic limit per sq in, tons	11.1	11
Elongation, per cent	14.05	18
Contraction, per cent	17.35	18

* Taken from the Mechanical World, London.

BY

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1. Shear

he internal stress in a body which resists the tendency of two adja- slide on each other, due to the action of two equal and parallel ces, called SHEARING- ng on opposite sides of shear. e *abcd* of Fig. 1 repre- ort simple beam of rial on which a suffi- applied, it will fail in EAR at *f* and *g*, as sliding on the sections at these points, be- pward force of the *S* and the downward load adjacent to *S*, ch it acts across the , is greater than the total SHEARING RESISTANCE of the section. ent over the entire length of the beam, and at any section is equal on at *S* minus the weight of the load between the reaction and the estion. In general, the VERTICAL SHEAR at any section of a beam vertical loads is equal to the algebraic sum of all the vertical forces acting on the beam on either side of the section.

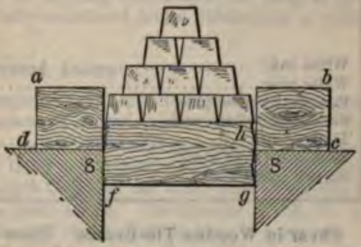
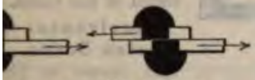


Fig. 1. Shearing-failure of Beam



Example of Single Shear

Single and Double Shear. A rivet connecting two bars under tension (Fig. 2) is subjected to a SHEARING-STRESS. If one section of

served by the area under stress. For material like wood, in whic
planes of weakness, tests must be made which take these into acc
direction of the force with respect to these planes must be considere
ing the SAFE WORKING STRESS from the tables.

Safe Working Stresses in Shear. Table I gives SAFE WORKIN
in SHEAR for those building materials usually subjected to such stre

Table I. Safe Working Stress in Shear for Building Mate

Material	Safe stress i sq in	
Cast iron (New York)	3 000	
Wrought iron	7 500	
Steel, bolts, rivets	10 000	
	With the grain	
White oak	200	
White pine	100	
Long-leaf yellow pine	150	
Short-leaf yellow pine	130	
Douglas fir	130	
Hemlock (New York)	40	
Spruce (New York)	50	

Shear in Wooden Tie-Beams. There are a few cases in archit
struction in which the weakness of wood in shear must be provide
one most frequently a
framing of the end
beams in wooden tru
Fig. 3 was mad
photograph of a
FAILURE of a tie-bea
thrust of the rafter.

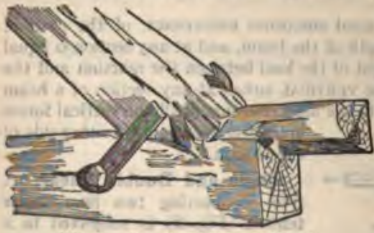


Fig. 3. Shearing-failure in Wood

Horizontal
Wooden Beams.
that shown in Fig
occurs in wood; but
wooden beams, the
which is less than about twenty times the depth, are liable
HORIZONTAL SHEAR along the middle, under about the same loads
the allowable working stresses in bending.

Shear at the End of a Tie-Beam. In the case of the truss-joi
the thrust S of the rafter tends to shear off the part $ABCD$ along t
which CD is the trace. This area under stress must offer a SHEAR
ANCE equal to the horizontal component H of the thrust S. The w
beam b, being fixed, formula (1) gives

$$H = (CD \times b)\, S_S \quad \text{or} \quad CD = H/bS_S$$

The shear being in the same direction as the grain of the wood, the
in Table I must be used.

ig. 4). The horizontal component of the thrust of a rafter is
long-leaf yellow pine tie-beam is 10 in wide. How far
extend beyond the

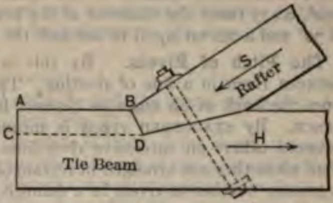


Fig. 4. Truss-joint

this case $H = 20\,000$
e I, $S_s = 150$ lb per

$D = \frac{20\,000}{10 \times 150} = 13.3$

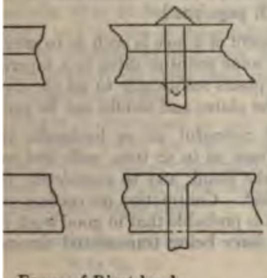
be made at least

constructed a large
st is generally taken
strap at the foot of
ld it in place. As
lder seldom act together, either the length *CD* on the tie-beam
long enough to resist the entire thrust, or the bolt or strap
without relying on the shearing resistance in the plane of *CD*.
ch joints is more fully considered under Subdivision 4, pages
s chapter.

2. Riveted Joints

s. Rivets almost exclusively are used in connecting the plates
h make up the members of framed steel construction.

ions. A rivet is a piece of cylindrical rod with a HEAD forged
usually with a slight taper at the other end of the SHANK. The
of the rivet is the length between the under sides of the heads
the thickness of the parts joined. The LENGTH (Table IV) of
l to the grip plus enough of the stock to form a head, and is
he end of the shank to the under surface of the head. The
E SHANK of a rivet is made equal to its NOMINAL DIAMETER, but rivets are driven into holes ⅛ in larger in diameter and upset by the driving so as to completely fill the holes. The shearing values and bearing values are based upon the NOMINAL AREA and not upon the area of the hole.



Forms of Rivet-heads

Riveting consists in heating the rivet to a welding-heat, passing it through holes in the parts to be joined and forging another head out of the projecting shank. This may be done by hand-hammering; but
ressed-air-operated hand-hammers or large riveting-machines
head and cause the shank to completely fill the hole by heavy
e.

Rivets. Rivets are made of soft steel and of wrought iron,
nerally used. The head may have any of the forms *shown in*
the first, called the BUTTON-*HEAD, is the standard for structural*
th *or COUNTERSUNK HEAD is used where it is necessary to have*
over a bearing-plate.

TANCE. By STAGGERED PITCH is meant the arrangement of rivets
between others on successive rivet-lines in order to decrease the se
than when they are arranged in rectangular rows, and at the same tim
a greater number of rivets in a definite area. The PITCH should not
less than three diameters of the rivet and the DISTANCE FROM THE ED
plate not less than one and one-half diameters, although it may be
to make the distance less when small angles are used. The pitch of
sunk rivets must be greater than that of button-head rivets becau
greater amount of material removed.

Punching Rivet-Holes. Rivet-holes are made with power-punch
SPACING is marked on the different parts to be fastened together by
wooden templates with holes drilled to locate the position of the rivet
the different parts are assembled, the holes are laid out by the same t
REGISTER, so that the rivets may be inserted without difficulty. I
makes a ragged hole. The flow of the metal under the great pressur
it and causes a loss in strength of from 11 to 33% as reported by W.
for soft steel. The injury may be removed by ANNEALING or by REAM
the injured part of the metal. Enlarging a $\frac{7}{8}$-in. hole by reaming
has been found to remove all the injurious effects of punching. On
practiced in the best work is to punch the holes $\frac{1}{16}$ in less in diamete
diameter of the rivets, and to ream them to a diameter $\frac{1}{16}$ in greater,
parts are assembled and bolted together. This removes the great
the injury from punching and corrects the alinement of the ho
Table XI, page 400, and Table I, page 702.)

Drift-Pins. When the alinement of a hole is such as to prevent the
of the rivet, it is the practice in some shops to drive in a tapered DRIF
distort the holes in some of the plates sufficiently to set the rivet. T
LOCAL STRESSES and injury to the plates and should not be permitted

Shop-Riveting is done with powerful air or hydraulic riveting
which may exert a pressure of from 30 to 50 tons, sufficient to upset
head on the projecting end of the shank and to completely fill the
though the alinement is imperfect. Contraction on cooling causes g
sure between the parts, so that it is probable that in good work the riv

of the rivet-head and tapping the other side with a light hammer. a slight slip may be felt. The loose rivets should be marked to be cut replaced. The inspector should also carefully check open holes left connections, and see that flattened and countersunk rivets are as called use such work may be done at less expense in the shop than in the ere it may cause delay.

'ailure of Riveted Joints may occur

TENSION, by the tearing of the plate through the line of rivets (Fig. 8).

SHEAR, by the cutting of the rivets (Fig. 7).

BEARING, by the crushing of the plate in front of the rivets, the split- he plate, or, in some cases, by the shearing out of the sections in front vets. In a careful design of a joint the strength against failure by each methods must be investigated (Fig. 6 and Fig. 9).

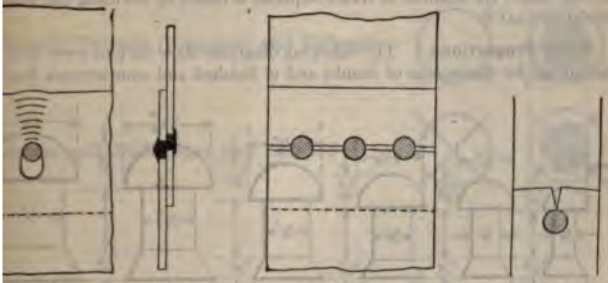


Fig. 6 Fig. 7 Fig. 8 Fig. 9

Figs. 6 to 9. Methods of Failure in Riveted Joints

Steps in the Design of any type of riveted joint are, (1) the selec- the size of the rivet to be used, (2) the determination of its shearing ring strength and the use of the smaller value of the two to divide into l load to be transmitted and thus determine the number of rivets, arrangement of the rivets in the plate and the investigation of its strength on at the dangerous section.

Size of Rivets is determined in part by SHOP-PRACTICE. Holes can- ounched in plates which are thicker than the diameter of the punch. owing table gives the size of rivets used with plates of different thickness. ecifications for structural work require all rivets to be ¾ in, except ick plates require larger ones.

Thickness of plates	Size of rivets
¼ to 7/16 in	⅝ in
½ to ⅝ in	¾ in
11/16 to 13/16 in	⅞ in
⅞ to 1 in	1 in

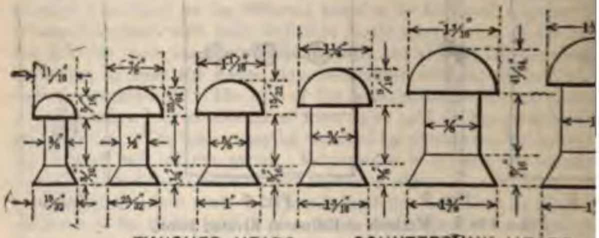
s II and III give the SHEARING and BEARING VALUES for different sizes in plates of different thickness for two values of working stresses each; 7 500 and 10 000 lb per sq in and bearing at 15 000 and 18 000 lb per Values for higher stresses can be figured by proportion from these tables. er stresses should be *used with wrought iron or in parts subjected to ds; the higher stresses where only constant or dead loads are present.*

The SHEARING VALUE is equal to the area of the rivet multiplied by th
stress; the BEARING VALUE is equal to the area of the projected surf
pressure multiplied by the working stress in bearing, or, if

t = the thickness of the plate;
d = the diameter of the rivet;
and S_b = the working stress in bearing;
then the bearing value $P = dtS_b$

The Shearing and Bearing Values may be taken directly from t
and if a rivet is in double shear, twice the quantity in the table is to b
its SHEARING VALUE. Quantities above the heavy broken lines ar
VALUES greater than the values in single shear, so that for these cond
number of rivets necessary in a joint required to transmit a certai
determined by dividing the load by the value in single shear. If riv
double shear, the number of rivets required is found by dividing the lo
BEARING VALUE.

Rivet-Proportions.* The following diagrams show various rivet-pr
including the dimensions of shanks and of finished and countersunk h



FINISHED HEADS

Diam head = 1½ diam of shank + ⅛" depth of head = ⁴⁵⁄₁₀₀ diam of head

COUNTERSUNK HEADS

Depth of head = ½ diam of shank. Bevel of head = 60°

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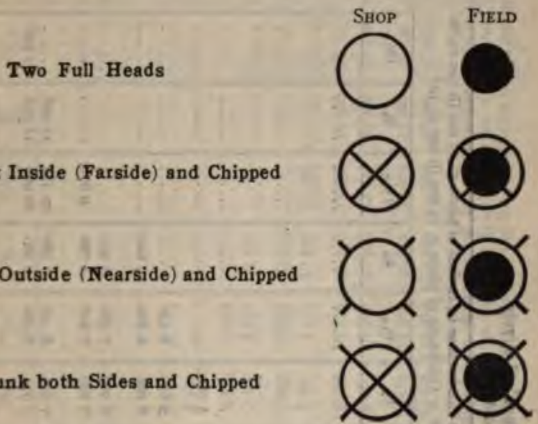
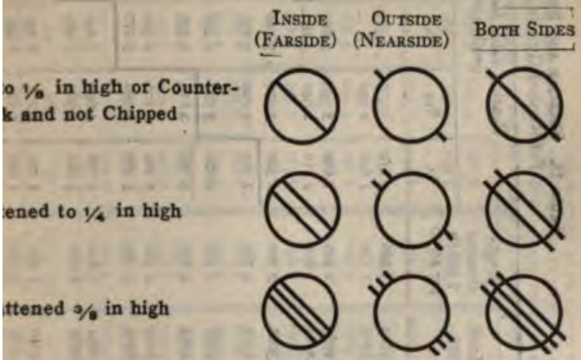
* These proportions vary slightly at different mills and in different han

onal Signs for Riveting. The following diagrams show some signs for riveting:


Shop
Field
Two Full Heads
unk Inside (Farside) and Chipped
nk Outside (Nearside) and Chipped
ersunk both Sides and Chipped



em, designed by F. C. Osborn, has for its foundation a diagonal cross t a countersink, a blackened circle for a field-rivet and a diagonal flattened head. The position of the cross with respect to the circle, ide, or on both sides, indicates the location of the countersink; and e number and position of the diagonal strokes indicate *the height* n of the flattened heads. *Any combination of field, countersunk* *d-head rivets liable to be used may be readily indicated by the* *ination of the above signs.*

Table II. Shearing and Bearing Values[*] of Rivets
For Riveted Girders and Wrought Iron

Diameter of rivet in inches		Area of rivet	Single shear at 7 500 lb per sq in	Bearing value for different thicknesses in inches of plate at 15 000 lb per sq in (=diameter of rivet × thickness of plate × 15 000 lb per sq in)										
Fractions	Decimals			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
3/8	0.375	0.1104	828	1 410	......	......	......	......	......	......	......	......	......	......
7/16	0.4375	0.1503	1 130	1 640	2 050	......	......	......	......	......	......	......	......	......
1/2	0.5	0.1963	1 470	1 880	2 340	2 810	......	......	......	......	......	......	......	......
9/16	0.5625	0.2485	1 860	2 110	2 640	3 160	3 690	......	......	......	......	......	......	......
5/8	0.625	0.3068	2 300	2 340	2 930	3 520	4 100	......	......	......	......	......	......	......
11/16	0.6876	0.3712	2 780	2 580	3 220	3 870	4 510	5 160	......	......	......	......	......	......
3/4	0.75	0.4418	3 310	2 810	3 520	4 220	4 920	5 630	6 330	......	......	......	......	......
13/16	0.8125	0.5185	3 890	3 050	3 810	4 570	5 330	6 090	6 860	7 620	......	......	......	......
7/8	0.875	0.6013	4 510	3 280	4 100	4 920	5 740	6 560	7 380	8 200	......	......	......	......
15/16	0.9375	0.6903	5 180	3 520	4 390	5 270	6 150	7 030	7 910	8 790	9 670	......	......	......
1	1.0	0.7854	5 890	3 750	4 690	5 620	6 560	7 500	8 440	9 380	10 310	11 250	......	......
1 1/16	1.0625	0.8866	6 650	3 980	4 980	5 980	6 970	7 970	8 960	9 960	10 960	11 950	12 950	......
1 1/8	1.125	0.9940	7 460	4 220	5 270	6 330	7 380	8 440	9 490	10 550	11 600	12 660	13 710	14 770
1 3/16	1.1875	1.1075	8 310	4 450	5 570	6 680	7 790	8 910	10 020	11 130	12 250	13 360	14 470	15 590

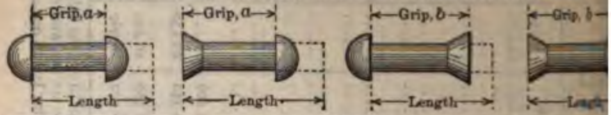
Fraction	Decimals	rivet	10 000 lb per sq in	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
3/8	0.375	0.1104	1 100	1 680	......	......	......	......	......	......	......	......	......	......
7/16	0.4375	0.1503	1 500	1 960	2 450	......	......	......	......	......	......	......	......	......
1/2	0.5	0.1963	1 960	2 240	2 800	3 370	......	......	......	......	......	......	......	......
9/16	0.5625	0.2485	2 480	2 520	3 150	3 790	4 420	......	......	......	......	......	......	......
5/8	0.625	0.3068	3 060	2 800	3 500	4 210	4 910	......	......	......	......	......	......	......
11/16	0.6875	0.3712	3 710	3 090	3 860	4 640	5 400	6 180	......	......	......	......	......	......
3/4	0.75	0.4418	4 420	3 370	4 210	5 060	5 890	6 740	7 580	......	......	......	......	......
13/16	0.8125	0.5185	5 180	3 650	4 560	5 480	6 390	7 300	8 210	9 120	......	......	......	......
7/8	0.875	0.6013	6 010	3 930	4 910	5 900	6 880	7 860	8 840	9 820	......	......	......	......
15/16	0.9375	0.6903	6 900	4 210	5 260	6 330	7 370	8 420	9 470	10 520	11 590	......	......	......
1	1.000	0.7854	7 850	4 500	5 620	6 750	7 860	9 000	10 120	11 240	12 370	13 500	......	......
1 1/16	1.0625	0.8866	8 860	4 780	5 970	7 170	8 360	9 560	10 750	11 940	13 140	14 340	15 530	......
1 1/8	1.125	0.9940	9 940	5 060	6 320	7 590	8 850	10 120	11 380	12 640	13 910	15 180	16 440	17 700
1 3/16	1.1875	1.1075	11 070	5 340	6 670	8 010	9 350	10 680	12 010	13 340	14 680	16 020	17 360	18 700

---

* Values for higher or lower unit stresses for shear or bearing to satisfy particular building laws can be figured by proportion from the table. See, also, paragraph on Working Stresses for Rivets, page 423. The Cambria Handbook (1912) gives values of from 12 000 to 20 000 and Carnegie's Pocket Companion (1915) values of from 14 000 to 24 000 lb per sq in for bearing for various structures. In the Boston building law 18 000 and in the New York City building law 20 000 lb per sq in are used.

Table IV. **Length of Field-Rivets for Various Grips. Length of Rive to Form Head**

American Bridge Company Standard. Dimensions in Inches



Grip *a*	Diameter ½	⅝	¾	⅞	1
½	1½	1¾	1⅞	2	2⅛
⅝	1⅝	1⅞	2	2⅛	2¼
¾	1¾	2	2⅛	2¼	2⅜
⅞	1⅞	2⅛	2¼	2⅜	2½
1	2	2¼	2⅜	2½	2⅝
⅛	2⅛	2⅜	2½	2⅝	2¾
¼	2¼	2½	2⅝	2¾	2⅞
⅜	2⅜	2⅝	2¾	2⅞	3
½	2⅝	2⅞	3	3⅛	3¼
⅝	2¾	3	3⅛	3¼	3⅜
¾	3	3¼	3⅜	3½	3⅝
⅞	3⅛	3⅜	3½	3⅝	3¾
2	3¼	3½	3⅝	3¾	3⅞
⅛	3⅜	3⅝	3¾	3⅞	4
¼	3½	3¾	3⅞	4	4⅛
⅜	3⅝	3⅞	4	4⅛	4¼
½	3¾	4	4⅛	4¼	4⅜
⅝	3⅞	4⅛	4¼	4⅜	4½
¾	4	4¼	4⅜	4½	4⅝
⅞	4⅛	4⅜	4½	4⅝	4¾
3	4⅜	4⅝	4¾	4⅞	5
⅛	4½	4¾	4⅞	5	5⅛
¼	4⅝	4⅞	5	5⅛	5¼
⅜	4¾	5	5⅛	5¼	5⅜
½	4⅞	5⅛	5¼	5⅜	5½
⅝	5	5¼	5⅜	5½	5⅝
¾	5⅛	5⅜	5½	5⅝	5¾
⅞	5¼	5½	5⅝	5¾	5⅞
4	5⅜	5⅝	5¾	5⅞	6
⅛	5⅝	5⅞	6	6⅛	6¼
¼	5¾	6	6⅛	6¼	6⅜
⅜	6	6¼	6⅜	6½	6⅝
½	6⅛	6⅜	6½	6⅝	6¾
⅝	6¼	6½	6⅝	6¾	6⅞
¾	6⅜	6⅝	6¾	6⅞	7
⅞	6½	6¾	6⅞	7	7⅛
5	6⅝	6⅞	7	7⅛	7¼
⅛	....	....	7⅛	7¼	7⅜
¼	....	....	7¼	7⅜	7½
⅜	....	....	7⅜	7½	7⅝
½	....	....	7⅝	7¾	7⅞
⅝	....	....	7¾	7⅞	8
¾	....	....	7⅞	8	8⅛
⅞	....	....	8	8⅛	8¼

Grip *b*	Diameter ½	⅝	¾	⅞
½	1⅛	1¼	1¼	1⅜
⅝	1¼	1⅜	1⅜	1½
¾	1⅜	1½	1½	1⅝
⅞	1½	1⅝	1⅝	1¾
1	1⅝	1¾	1¾	1⅞
⅛	1¾	1⅞	1⅞	2
¼	1⅞	2	2	2⅛
⅜	2	2⅛	2⅛	2¼
½	2⅛	2¼	2⅜	2⅜
⅝	2¼	2⅜	2½	2½
¾	2½	2⅝	2¾	2¾
⅞	2⅝	2¾	2⅞	2⅞
2	2¾	2⅞	3	3
⅛	2⅞	3	3⅛	3⅛
¼	3	3⅛	3¼	3¼
⅜	3⅛	3¼	3⅜	3⅜
½	3¼	3⅜	3½	3½
⅝	3⅜	3½	3⅝	3⅝
¾	3½	3⅝	3¾	3¾
⅞	3⅝	3¾	3⅞	3⅞
3	3⅞	4	4	4⅛
⅛	4	4⅛	4⅛	4¼
¼	4⅛	4¼	4¼	4⅜
⅜	4¼	4⅜	4⅜	4½
½	4⅜	4½	4½	4⅝
⅝	4½	4⅝	4⅝	4¾
¾	4⅝	4¾	4¾	4⅞
⅞	4¾	4⅞	4⅞	5
4	4⅞	5	5	5⅛
⅛	5⅛	5¼	5¼	5⅜
¼	5¼	5⅜	5⅜	5½
⅜	5½	5⅝	5⅝	5⅝
½	5⅝	5¾	5¾	5¾
⅝	5¾	5⅞	5⅞	5⅞
¾	5⅞	6	6	6
⅞	6	6⅛	6⅛	6⅛
5	6⅛	6¼	6¼	6¼
⅛	....	....	6⅜	6⅜
¼	....	....	6½	6½
⅜	....	....	6⅝	6⅝
½	....	....	6⅞	6⅞
⅝	....	....	7	7
¾	....	....	7⅛	7⅛
⅞	....	....	7¼	7¼

For weight of rivets, see page 1442.

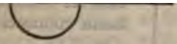


Fig. 10. Lap-joint

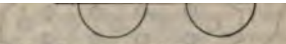


Fig. 11. Butt-joint with Single Cover-plate

g of Tie-Bars. Tie-bars may be spliced by a LAP-JOINT (Fig. 10); T-JOINT with a single cover-plate (Fig. 11); or by a BUTT-JOINT cover-plates (Fig.

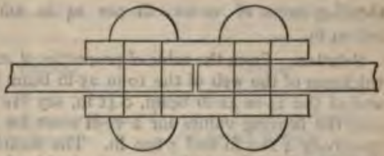
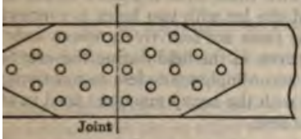


Fig. 12. Butt-joint with Two Cover-plates

utt-Joint is symmet- more efficient than because of the absence dency to bend when ad. The net area of -plates at the section e rivets at the end of plate must be equal to ea of the main plate through the rivets at the end of the cover-plate. ows a better arrangement of rivets than that in Fig. 13, because less moved at the critical section of the cover-plates. In some cases it may be necessary to make the aggregate thickness of the cover-plates greater than the thickness of the main plates.

A joint with one line of rivets is said to be SINGLE-RIVETED, one with two lines DOUBLE-RIVETED, and one with more than two lines, CHAIN-RIVETED.



over-plate. Six Rivets at Critical Section

e 2. It is required to determine the number of rivets in the splice ½-in tie-bar which is subject to a tensile force of 65 000 lb.

1. Assuming that the load is constant, the stresses in Table III may Assuming, also, a lap-joint like that in Fig. 15, and ¾-in rivets, the

greater. The length of the horizontal or third side of the right-angled
having an hypotenuse of 2.25 in and a vertical altitude of 1.5 in,

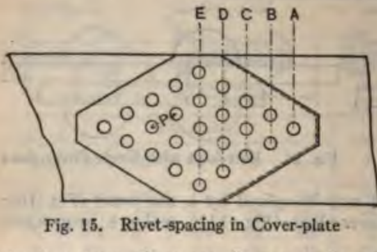


Fig. 15. Rivet-spacing in Cover-plate

which requires that thi
ED, etc., be 1.75 in, if
in multiples of $\frac{1}{4}$ in.

Floor-Beam Con
The two following ex
lustrate common types
beam connections.

Example 3. It is re
determine the numbe
rivets to connect a
I beam supporting 24
a 15-in 42-lb **I** beam
shearing-stress of 10 000 lb per sq in and a bearing-stress of
per sq in.

Solution. From the table of properties of standard **I** beams, pages
thickness of the web of the 10-in 25-lb beam is found to be 0.31 in,
and of the 15-in 42-lb beam, 0.41 in, say $\frac{7}{16}$ in. Referring to Tabl
419, the bearing values for a $\frac{3}{4}$-in rivet for these thicknesses of we
spectively 4 210 lb and 5 890 lb. The shearing value of the rivet i
The rivets in the 10-in beam are in double shear; hence the bearing
erns. The number of rivets, then, is 12 000, the end-reaction, divided
or 3. For the 15-in beam the shearing value is less, and the numbe
required is 12 000 divided by 4 420, or 3. Hence two standard
angles, 6 by 4 by $\frac{3}{8}$ in and 5 in long, may be used. Each has three h
leg and two in the other. The leg with three holes is placed on the
with the rivets in double shear, and the leg with two holes is connec
15-in beam; thus, in the latter case there are four rivets where only
required for strength. They are driven in the field during the erecti
structure and the working stress is accordingly made less in most spe
because of the better work possible with the heavy machines used in
than with the tools available in the field.

Example 4. It is required to determine the number of $\frac{3}{4}$-in r
4 by 4 by $\frac{1}{2}$-in angle-bracket attached to an 18-in 55-lb beam and
ing a 10 by 12-in wooden beam on which there is a load of 18 000 lb.

Solution. The rivets are in single shear with a shearing-resistance o
taken from Table III. The thickness of the web of the **I** beam is $\frac{7}{16}$
a bearing value of 5 890 lb. Dividing 9 000 lb, the end-reaction, by
the controlling value, we find that two rivets are insufficient. The br
be fastened with three $\frac{3}{4}$-in rivets with a spacing of 4 in. Two
are sufficient to hold the bracket.

Rivets in Plate Girders. The methods of determining the rivet
and box girders are given in Chapter XX.

Bending Stress in Rivets. While the BENDING STRENGTH of P
joints of articulated trusses is always investigated, this is never do
case of RIVETS. A hot rivet properly driven is, when cold, under
stress which is nearly equal to the elastic limit of the material. T
great pressure between the plates and a consequent frictional res
movement, which, under the usual conditions, equals the allowed shea
on the rivet; and so, until an INITIAL SLIP occurs, there can be
STRESSES in the rivet. In the case of very long rivets driven in

conditions of service, as in the floor-systems of bridges. Table III r ordinary structural work made under the conditions governing in iop-practice. For comparison, the following stresses taken from the ons of Theodore Cooper for Steel Railroad Bridges and Steel Highway re given:

fication for	Allowable stresses on rivets, lb per sq in	
	Bearing	Shear
lroad bridges	15 000 (12 000 on floors) 22 500 for laterals	9 000 (7 200 on floors) 13 500 for laterals
hway bridges	18 000 (14 400 on floor-beams) 27 000 for laterals	10 000 (8 000 on floor-beams) 14 000 for laterals

ven in the field are allowed two-thirds the value of shop-driven rivets.

3. Strength of Pins in Trusses*

Pins. In the design of the PINS at the joints of trusses the stresses in ARING FLEXURE or BENDING must be investigated.

earing-Force at any section of the pin is the algebraic sum of all cting on the pin on either side of the section. The stress is considered ormly distributed over the cross-section of the pin. When the forces in the same plane they must be resolved into vertical and horizontal s and the resultant of these components taken as the shear at any ction. This may be done by the principles of GRAPHIC STATICS, or OMETRICAL and ALGEBRAICAL METHODS, the graphic method being, the more rapid.

aring Area on the pin is taken as the PROJECTION OF THE AREA OF zontal components and these component forces in the two planes treated se rately. The resultants in both planes at any section may be combined and a single resultant force acting on the section obtained, and also the conseque stresses due to it.

Table V. Shearing and Bearing Values of Pins for One-Inch Thicknes of Plate, in Pounds per Square Inch

Diameter of pin, in	Area of pin, sq in	Bearing value at 12 000 lb per sq in, lb	Single shear 7 500 lb per sq in, lb	Diameter of pin, in	Area of pin, sq in	Bearing value at 12 000 lb per sq in, lb	Single shear 7 5 lb per sq tons
1	0.785	12 000	5 890	4	12.57	48 000	47.
1⅛	0.994	13 500	7 455	4⅛	13.36	49 500	50.
1¼	1.227	15 000	9 202	4¼	14.19	51 000	53.
1⅜	1.485	16 500	11 132	4⅜	15.03	52 500	56.
1½	1.767	18 000	13 252	4½	15.90	54 000	59.
1⅝	2.074	19 500	15 555	4⅝	16.80	55 500	63.
1¾	2.405	21 000	18 037	4¾	17.72	57 000	66.
1⅞	2.760	22 500	20 707	4⅞	18.67	58 500	70.
2	3.142	24 000	23 565	5	19.64	60 000	73.
2⅛	3.547	25 500	26 600	5⅛	20.63	61 500	77.
2¼	3.976	27 000	29 820	5¼	21.65	63 000	81.
2⅜	4.430	28 500	33 225	5⅜	22.69	64 500	85.
2½	4.909	30 000	36 817	5½	23.76	66 000	89.
2⅝	5.412	31 500	40 590	5⅝	24.85	67 500	93.
2¾	5.940	33 000	44 550	5¾	25.97	69 000	97.
2⅞	6.492	34 500	48 690	5⅞	27.11	70 500	101.
			tons				
3	7.069	36 000	26.5	6	28.27	72 000	106
3⅛	7.670	37 500	28.7	6⅛	29.46	73 500	110
3¼	8.296	39 000	31.0	6¼	30.68	75 000	115
3⅜	8.946	40 500	33.5	6⅜	31.92	76 500	119
3½	9.621	42 000	36.0	6½	33.18	78 000	124
3⅝	10.32	43 500	38.7	6⅝	34.47	79 500	129
3¾	11.05	45 000	41.4	6¾	35.79	81 000	134
3⅞	11.79	46 500	44.2	6⅞	37.12	82 500	139

In the Method of Moments a section is taken at each force in successio and the moment of the forces about a point in the section found, due conside ation being given to the direc tion of turning. This is do at each force on one side of th pin, if the bars are arrange symmetrically, and in both th vertical and horizontal plane Inspection of the results wi usually indicate which sectio has the GREATEST RESULTANT MOMENT when the horizontal and vertic components, H and V, are combined. This is done by using the formu $R^2 = H^2 + V^2$ since, graphically, the resultant R is the diagonal of the recta

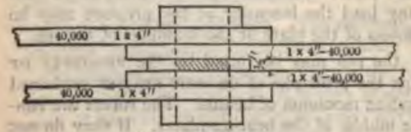


Fig. 16. Pin-joint

**e VI. Maximum Bending Moments in Inch-Pounds to be Allowed
Pins for Maximum Fiber-Stresses of 15 000, 20 000 and 22 500
Pounds per Square Inch**

Moment for S = 15 000 in-lb	Moment for S = 20 000 in-lb	Moment for S = 22 500 in-lb	Diameter of pin, in	Moment for S = 15 000 in-lb	Moment for S = 20 000 in-lb	Moment for S = 22 500 in-lb
1 470	1 960	2 210	4	94 200	125 700	141 400
2 100	2 800	3 140	4⅛	103 400	137 800	155 000
2 880	3 830	4 310	4¼	113 000	150 700	169 600
3 830	5 100	5 740	4⅜	123 300	164 400	185 000
4 970	6 630	7 460	4½	134 200	178 900	201 300
6 320	8 430	9 480	4⅝	145 700	194 300	218 500
7 890	10 500	11 800	4¾	157 800	210 400	236 700
9 710	12 900	14 600	4⅞	170 600	227 500	255 900
11 800	15 700	17 700	5	184 100	245 400	276 100
14 100	18 800	21 200	5⅛	198 200	264 300	297 300
16 800	22 400	25 200	5¼	213 100	284 100	319 600
19 700	26 300	29 600	5⅜	228 700	304 900	343 000
23 000	30 700	34 500	5½	245 000	326 700	367 500
26 600	35 500	40 000	5⅝	262 100	349 500	393 100
30 600	40 800	45 900	5¾	280 000	373 300	419 900
35 000	46 700	52 500	5⅞	298 600	398 200	447 900
39 800	53 000	59 600	6	318 100	424 100	477 100
44 900	59 900	67 400	6⅛	338 400	451 200	507 600
50 600	67 400	75 800	6¼	359 500	479 400	539 300
56 600	75 500	84 900	6⅜	381 500	508 700	572 300
63 100	84 200	94 700	6½	404 400	539 200	606 600
70 100	93 500	105 200	6⅝	428 200	570 900	642 300
77 700	103 500	116 500	6¾	452 900	603 900	679 400
85 700	114 200	128 500	6⅞	478 500	638 000	717 800

rks. The following is the formula for flexure, $M = SI/c$, with the reductions adapt it to a beam of circular section:

$$M = S\pi d3/32 = SAd/8$$

M = the moment of forces for any section through the pin;
S = the stress per sq in in extreme fibers of pin at that section;
A = the area of the section;
d = the diameter;
π = 3.14159.

orces are assumed to act in a plane passing through the axis of the pin.
bove table gives the values of M for different diameters of pin, and for three
l S.

maximum value of M is known, an inspection of the table will show what the
r of the pin must be so that S will not exceed 15 000, 20 000, or 22 500 lb, as
irements of the case may be.

nd V. Example 6 illustrates the *method for the condition of* INCLINED acting *on the pin. In Example 5 the same method is employed* to deter- *size of the pin in a simple joint.*

find the bending moment on the pin the forces are considered concent
the middle of the bars and moments taken about sections through th
The moment at the section through the second bar is 40 000 lb × 1 i
to 40 000 in-lb. If moments are taken about a point between the in
the same result is obtained. From Table VI it is found that a 2¾-i
20 000 lb per sq in is sufficient. From Table V the bearing value of
pin is found to be only 33 000 lb at a stress of 12 000 lb per sq in, whic
it necessary to increase the size of the pin to 3⅝ in. The shearing
this pin is 67 000 lb. In this case the diameter of the pin is determ
the bearing-stress, but it is necessary to investigate the other stress
sure of the correct size, especially in case of heavy bearing-plates.

Bending Moments on Pins. The finding of the BENDING MOMEN
the forces acting on a pin is usually the most difficult part of the work
mining its proper size. In the case of a simple pin, properly packed
in the plane of the forces acting on it, the GREATEST MOMENT is usually
uct obtained by multiplying the outer force by the central distance
the outer bars; but when the forces act in several planes the work
complicated. The GRAPHICAL METHOD illustrated in the solution of
following examples has some advantages; but the METHOD OF MOMENT
at the end of the solution of the first example is equally rapid in
hands and capable of greater refinement in the results.

Example 6. It is required to find the bending moment on the p
joint, one-half of which is shown in Fig. 17. The bars are each 1
the channel of the vertical member ½ in thick and the center of th
is ¾ in from the center of the channel.

Solution. Since the joint is symmetrical it is necessary to construct
half of the force-diagram and equilibrium-polygon which really
the joint. From the conditions of equilibrium of forces, the verti
ponent of the inclined force is upward, and equal to the sum of the d
forces, 34 000 lb; and its horizontal component acts with the 60 000

s the pole of the stress-diagram, the pole-distance being 20 000 lb. principles of graphics the bending moment at any point on the pin to the intercept between the proper ray of the equilibrium-polygon losing line, multiplied by the pole-distance. To complete the figures, and 0–4 are drawn from O, and from c cd is drawn parallel to 0–2, de o 0–3, ef parallel to 0–4 and fk parallel to 0–1. In the same way wn parallel to 0–5, st to 0–6 and tv to 0–1. Then according to the inciples, the moment at any section due to the forces in the horizontal proportional to the ordinate at that section drawn from the line AB e $cdefk$ bounding the equilibrium-polygon; and the moments due to cal forces are proportional to the ordinates drawn to the line $rstv$, the l value being the length of the ordinate times 20 000, the pole-distance.

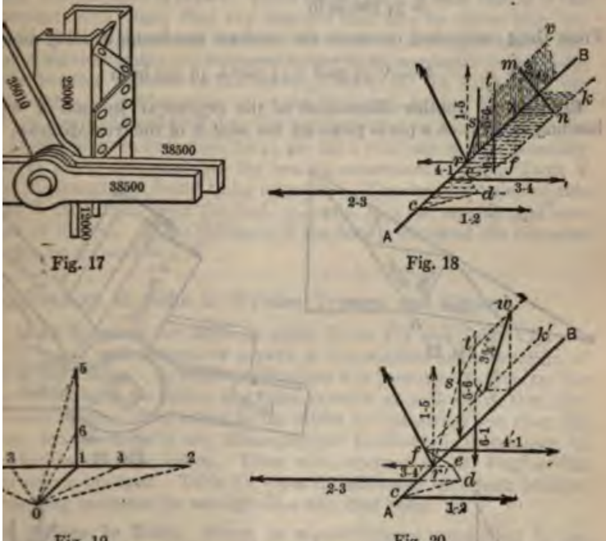


Fig. 17

Fig. 18

Fig. 19

Fig. 20

Figs. 17 to 20. Pin-joint and Moment-diagrams

th moments are present, the resultant or true moment is proportional potenuse of the right-angled triangle having for its sides the ordinates o planes at the point in question. At X this is shown by the line mn. sures 2.42 in. and being the longest diagonal or hypotenuse that can oppose each other are placed together. It may be further reduced by the outside bar one-half the thickness of the main horizontal bars.

To check by the METHOD OF MOMENTS the value of the maximum moment obtained by the GRAPHIC METHOD for the first arrangement, ments of the forces in the horizontal plane are taken about r. This gi

$$M_h = 38\ 500 \text{ lb} \times 3.0 \text{ in} + 38\ 500 \text{ lb} \times 1.0 \text{ in} - 60\ 000 \text{ lb} \times 2.0 \text{ in}$$
$$= 34\ 000 \text{ in-lb},$$

which is the value of the moment in the horizontal plane across the the pin.

In the vertical plane moments are taken about a point t, giving

$$M_v = 34\ 000 \text{ lb} \times 1.5 \text{ in} - 22\ 000 \text{ lb} \times 0.75 \text{ in}$$
$$= 34\ 500 \text{ in-lb}$$

From these component moments the resultant maximum bending mo

$$M = \sqrt{34\ 000^2 + 34\ 500^2} = 48\ 400 \text{ in-lb}$$

Example 7. Another illustration of the GRAPHICAL METHOD of fi bending moment on a pin is given for the joint A of the truss-diagram

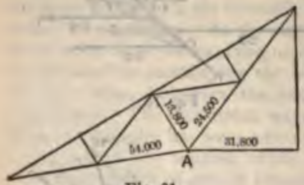


Fig. 21

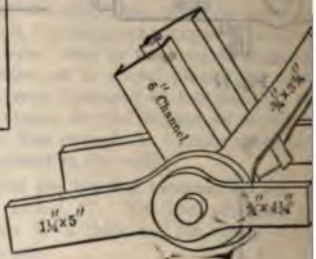


Fig. 22

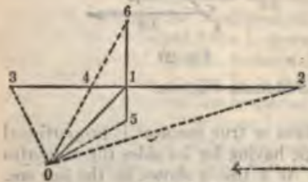


Fig. 23

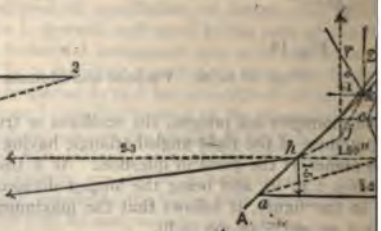


Fig. 24

Figs. 21 to 24. Force-polygons and Equilibrium-polygons for Bending Mon a Pin

Fig. 21. Fig. 22 shows the arrangement and size of the members. Th given in Fig. 21 are for one-half the number of members at the join Example 6, the symmetrical arrangement makes it unnecessary to

distance is then laid off at an angle of 45° and equal to 20 000 lb to scale of forces. The pole o is then joined with 2, 3, 4 and 1. Then , *ab* is drawn parallel to o–2, *bc* to o–3, *cd* to o–4 and *de* to o–1. In way the line *hjkB* is drawn. From inspection it is seen that *hb* is the tercept, even longer than any diagonal that may be drawn from the s of the horizontal and vertical intercepts at any point along *AB*. ne scale that makes o–1 represent 20 000 lb, *hb* represents 31 800 in-lb; the bending moment on the pin is 31 800 in-lb. In Table VI a pin diameter, at a fiber-stress of 20 000 lb per sq in, has an allowable f 35 500 in-lb, and in Table V a bearing value on 1 in of 31 500 lb. 31 800 lb on ¾ in is equal to 42 400 for a 1-in bar; so it is necessary arger pin to accommodate the bearing requirement. From Table V in in diameter is found to be necessary. The shearing value of this oo lb more than twice the load, so, again, it is the bearing that con-ize of the pin. If the thickness of the bars is increased the diameter may be reduced to 3 in.

. Strength of Bolts in Wooden Trusses and Girders

orking Stresses for Bolts on which Table VII and Table VIII are are based on a FACTOR OF SAFETY of five applied to the average of s on dry timber. In some specifications it is permitted to increase the RESSURE between timber and bolts as much as 50% above that per- short struts. The values in the tables are somewhat less than the arge trusses made at the Massachusetts Institute of Technology, in ld indicate as safe values. These were reported in the Engineering ovember 17, 1900. Table IX gives the allowable maximum tension, bending moments for wrought-iron and steel bolts.

of Stress in Bolts. BOLTS in wooden trusses are subject to the ls of stress as the RIVETS and PINS in steel structures. When the ned are less than 2 in thick and the bolts are tightly drawn up so as considerable frictional resistance between the pieces, the bolts are ned to resist the total force in SHEAR and in BEARING. When the

(1) Bolts in tie-beams, thin pieces.
(2) Bolts in girders to support brackets.
(3) Bolts as pins in the joints of trusses.
(4) Bolt-and-strap joints in trusses.
(5) Bolts under tension to hold the foot of a rafter.

(See, also, "Joints in Wooden Trusses," Chapter XXVIII, pages 114

Case 1. Bolts in Tie-Beams, Thin Pieces. Tie-beams of wood when longer than 30 ft, are usually made up of a number of pieces. struction is cheaper than the use of a single stick. Two-inch plan together are generally used. The location of the joints in the courses and the number and size of the bolts are the special considerations in of such a joint. In general, the joints in adjacent courses are pla apart as possible and not more than two joints are placed opposite in the same section. The simplest case is that of a plain FISH-PLATE a common BUTT-JOINT with two cover-plates as shown in Fig. 12. Th of BOLTS for such a joint is found in the same way as the number of steel tie-bars. The bolts must be spaced as required in the second colu each timber in Table VII, to provide against shearing in front of the

Table VII.* Safe Bearing Value of Bolts per Inch of Length Par the Grain in Timber and Distance from Center to Center of B or to End of Timber

— — Diameter of bolt, in	Long-leaf yellow pine		White pine and short-leaf yellow pine		Douglas fir		Whi
	Bearing at 1 400 lb per sq in, lb	Distance, in	Bearing at 1 100 lb per sq in, lb	Distance, in	Bearing at 1 200 lb per sq in, lb	Distance, in	Bearin at 1 40 lb per sq in, lb
¾	1 050	4½	825	5¼	900	4¼	1 050
⅞	1 225	5	960	5¾	1 050	5	1 225
1	1 400	5¾	1 100	6½	1 200	5½	1 400
1⅛	1 575	6½	1 237	7½	1 350	6¼	1 575
1¼	1 750	7	1 375	8	1 500	7	1 750
1⅜	1 925	7¾	1 512	9	1 650	7¾	1 925
1½	2 100	8½	1 650	9¾	1 800	8½	2 100
1¾	2 450	10	1 925	11½	1 950	9¼	2 450
2	2 800	11½	2 200	13	2 400	11¼	2 800
2¼	3 150	12¾	2 475	14¾	2 700	12½	3 150
2½	3 500	14¼	2 750	16¼	3 000	14	3 500
2¾	3 850	15¼	3 025	18	3 300	15½	3 850
3	4 200	17	3 300	19	3 600	17	4 200

The distance from the end is equal to the diameter of the bolt plus the lengt twice the SHEAR is equal to the BEARING VALUE of the bolt against the end-fib

---

* When the effect of the inclined surfaces upon the unit stresses is taken int the formula for the normal intensity of stress for cylindrical pins or bolts, given XXVIII, page 1138, may be used. This formula will give lower values th Table VII.

VIII.* Safe Bearing Value of Bolts per Inch of Length Across the Grain in Timber

eter lt,	Long-leaf yellow pine, lb	Short-leaf yellow pine and Douglas fir, lb	White pine, lb	White oak, lb
	262	187	150	375
	306	218	175	437
	350	250	200	500
	394	281	225	562
	437	312	250	625
	482	343	275	687
	525	375	300	750
	612	437	350	875
	700	500	400	1000

IX. Maximum Allowable Tension, Shear and Bending Moment for Wrought-Iron and Steel Bolts

	Wrought iron			Steel		
Net area, sq in	Tension at 12 000 lb per sq in, lb	Shear at 7 500 lb per sq in, lb	Bending moment at 15 000 lb per sq in, in-lb	Tension at 16 000 lb per sq in, lb	Shear at 10 000 lb per sq in, lb	Bending moment at 20 000 lb per sq in, in-lb
0.302	3 620	3 310	620	4 830	4 420	830
0.420	5 040	4 510	980	6 720	6 010	1 310
0.550	6 600	5 890	1 470	8 800	7 850	1 960
0.694	8 328	7 460	2 100	11 100	9 940	2 800
0.893	10 716	9 200	2 880	14 290	12 270	3 830
1.057	12 680	11 140	3 830	16 910	14 850	5 100
1.295	15 540	13 250	4 970	20 720	17 670	6 630
1.746	20 930	18 040	7 890	27 910	24 050	10 500
2.302	27 620	23 560	11 800	36 830	31 420	15 700
3.023	36 280	29 820	16 800	48 370	39 760	22 400
3.719	44 630	36 820	23 000	59 510	49 090	30 700
4.620	55 430	44 550	30 600	73 910	59 400	40 800
5.428	65 140	53 010	39 800	86 850	70 690	53 000
6.510	78 120	62 220	50 600	104 160	82 960	67 400

e 8. A typical tie-beam used as a lower chord of a Howe truss is Fig. 25. It is 50 ft long, of Douglas fir and subject to the tension in ent panels shown in the figure.

n. The thickness of the plank is drawn out of scale in the figure to joints more clearly. The black circles show the vertical tension-rods, nearly cut the middle plank in two that *it is not considered a part of* le member. *The arrangement of the planks and the lengths to be* *t be determined for each case. In the one shown there is but one* *e middle panels where there is the greatest tension.* The distance

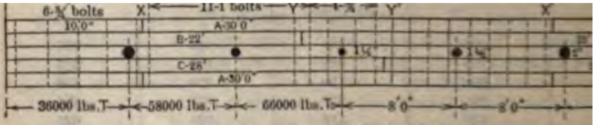


Fig. 25. Plan of Built-up Tie-beam

joints in A and A' this must be transmitted to B and C in order t
point X.

Assuming that 29 000 lb, one-half the tension, is carried on plan
transmitted to C by the shear and bearing on the bolts, and dividi
7 850 lb, the allowable shear on a 1-in bolt, four bolts are found to be
But the bearing value of a 1-in bolt in Douglas fir 2 in thick, is onl
which makes twelve bolts necessary. These are required in the dis
12 ft.

From the distances in Table VII, it is found that the end-bolts mus
from the ends of the planks, say 6 in; this leaves 11 ft, in which dist
bolts are to be arranged. If four bolts are placed in pairs, two at ea

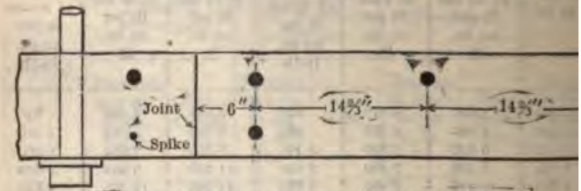


Fig. 26. Elevation of Beam Opposite X of Fig. 25

shown in Fig. 26, the intermediate spaces are 14¾ in. The bolts bind
together better if they are staggered, as indicated in Fig. 26, and not
the middle line.

The number of bolts mentioned is sufficient to make the splice st be kept within the values given in Table VII for the timber used. case shown in Fig. 28 the BENDING MOMENT in pound-inches is

$$M = SL/2 \quad \text{or} \quad M = S'L/2$$

is the larger. B and L are measured in inches and S in pounds.

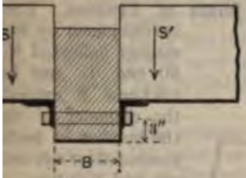


Fig. 27

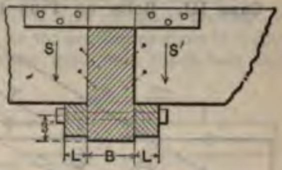


Fig. 28

Figs. 27 and 28. Bolts Supporting Brackets on Girders

9. For the construction shown in Fig. 27 it is required to determine er and size of bolts, the Douglas fir girder being 8 by 14 in, with a ft, and the Douglas fir joists 3 by 12 in, with a span of 20 ft, center to girders. The floor-load, including the floor, is 60 lb per sq ft. The 4 by 3½ by ⅜ in.

. The floor-area supported by the girder is 14 by 20 ft. At 60 lb the load is 14 × 20 × 60 = 16 800 lb. The load S, on one side, is

bolt has a shearing-value of 4 420 lb. Hence two bolts are necessary the shearing condition. The bearing value of the bolt in the wood, grain, is, from Table VIII, 187 lb per inch of length, or 1 496 lb for of the girder. The number of bolts required, then, is 16 800 divided r approximately 11, which gives a spacing of about 15 in.

10. In the construction shown in Fig. 28, the girder is 6 by 14 in, of r and has a span of 12 ft. The joists are 2 by 12 in and have an 18-ft er to center of girders. The floor-load is 65 lb per sq ft. There are trips on the sides of the girder. The distance L is 3 in. It is required number and size of bolts to be used.

. The total load on the girder is

$$12 \times 18 \times 65 = 14\,040 \text{ lb}$$
$$S = 7\,020 \text{ lb}$$

ring load per inch of thickness of the girder is

$$\frac{14\,040}{6} = 2\,340 \text{ lb}$$

ding moment on one side of the girder is

$$\frac{7\,020 \times 3}{2} = 10\,530 \text{ in-lb}$$

orce S acts at the center of pressure on the bracket-strip, 1½ in from f the girder.

ar is 7 020 lb, which requires two ¾-in steel bolts at 4 420 lb for one n Table IX.

ring (Table VIII) on a ¾-in bolt is 187 lb per inch of length; therefore thirteen bolts for bearing.

The allowable bending moment on a ¾-in steel bolt is 830 in-lb, from To take care of the 10 530 in-lb requires thirteen bolts. A ⅞-in ste an allowable bending moment of 1 310 lb-in, making eight of them The 3 by 4-in pieces may be held in place by thirteen ¾-in bolts sp on centers, if two of them are placed 6 in from the ends.

Case III. Bolts as Pins in the Joints of Trusses. Fo STRUTS joined by B manner indicated i 30 and 31 and thickness *B* excee the diameter of t the number of bol computed for SHEAR ING and FLEXURE.

For any of these forces are as follows

The single shea

On the sections and *B′* (Fig. 30)

The bearing on t inch of length = *S*/

The greater is to be

The bending momen

on the assumption o UOUS BEAM, uniform

If there are more one, the quantities by the above formulas are to be divided by the number of bolts to fin to be taken care of by one bolt.

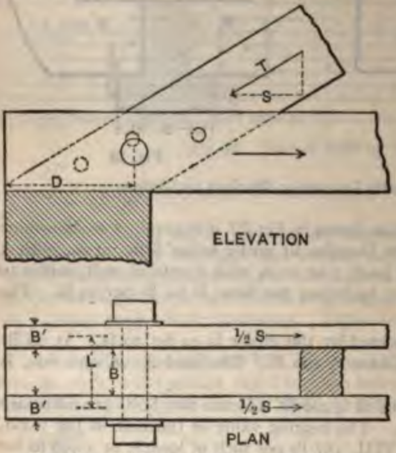


Fig. 29. Bolt through Rafter and Tie-beam

In Fig. 29, *S* is the horizontal component of the thrust *T*.

Example 11. It is required to determine the diameter of a bolt like that shown in Fig. 29. The rafter is 6 by 10 in, of Douglas fir, the tie-beams 3 by 10 in, of the same material, the thrust in the rafter 30 000 lb, and its inclination 30°.

Solution. The horizontal component of 30 000 lb at 30° is practically 26 000 lb. Then $S = 26\,000$ lb and the shear = 13 000 lb. $B = 6$ in and $L = 9$ in.

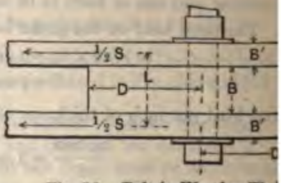


Fig. 30. Bolt in Wooden Tie-

Bearing per inch of length on the bolt = 26 000/6 = 4 333 l
Bending moment = 26 000 × 9/12 = 19 500 in-lb

In Table IX, a 1⅜-in steel bolt is found to be necessary to resist 13 000 lb, and a 2¼-in bolt for a bending moment of 19 500 in-lb. 4 333 lb end-bearing pressure on 1 in a larger bolt is required than Table VII. Dividing 4 333 by 1 200, the allowable bearing on Do 3⅝-in bolt is found to be necessary. This is larger than it is desira so the joint must be redesigned with a view to reduce the bearing f an 8 by 8-in strut and 4 by 8-in tie-beams are used, B becomes 2 in. This gives

pressure = 26 000/8 = 3 250 lb per inch of length of the bolt
g moment = 26 000 × 12/12 = 26 000 in-lb

ear at the section on one side of the strut is the same as before. ble VII it is found that a 2¾-in bolt is large enough to provide for and that a 2½-in bolt is sufficient for the bending as given in Table if an 8 by 8-in strut is used, there must be a 2¾-in bolt and D must be 15½ in (Table

12. For the same construc- ig. 29 and the same con- the first part of Example uired to determine the size s when it is necessary to

The shear, bearing, and ment are the same as in , but because there are each quantity is divided by 3 to determine the force resisted by

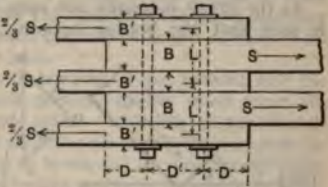


Fig. 31. Bolts in Wooden Tie-beam

3 000/3 = 4 333 lb and requires a ¾-in steel bolt (Table IX)
4 333/3 = 1 444 lb and requires a 1¼-in bolt (Table VII)
moment = 19 500/3 = 6 500 in-lb, and requires a 1½-in steel bolt

se the bending moment determines the size of the bolts, which may as shown by the dotted circles in Fig. 29.

13. It is required to determine the diameter of the bolt for the shown in Fig. 30, in which the inner beam is of Douglas fir and section, and the outer beams 3 by 8 in, the tension being 24 000 lb.

$$S = 24\,000; \quad B = 6 \text{ in}; \quad L = 9 \text{ in}.$$

e shear on the bolt = 24 000/2 = 12 000 lb
ing-pressure per inch of length of bolt = 24 000/6 = 4 000 lb
ing moment = 24 000 × 9/12 = 18 000 in-lb

ble IX a 1¼-in steel bolt is found sufficient to resist the shear, and large enough to resist the bending. In Table VII the largest bolt 3 in, is too small in bearing value. Dividing the load to be resisted ves 3⅓ in, as the diameter necessary to resist the bearing. The must be 4 000/(2 × 130) + 3⅓ in or 18¾ in.

14. If two bolts are used, one behind the other, it is required to he diameter of the bolt that should be used, the conditions and g the same as in Example 13.

Dividing the quantities obtained in Example 13 by 2,

gle shear = 6 000 lb and requires a ⅞-in steel bolt
aring = 2 000 lb and requires a 2-in bolt
nding moment = 9 000 in-lb and requires a 1¾-in steel bolt

able bearing on a 1¾-in bolt is (2½ %) less than the required amount, general, since the other requirements are more than satisfied, the would be used. For the 1¾-in bolt, the distance D is 9¼ in. The the bolts may be increased somewhat beyond the value given in to resist the full force in the direction of the STRAP.

As the STRAP is usually not more than from ½ to ¾ in thick,
is such that the bearing between it and the rafter is small

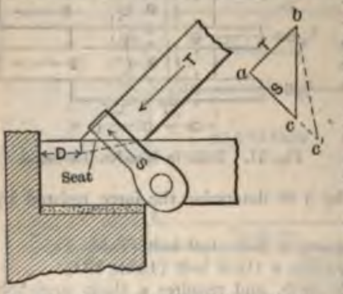


Fig. 32. Strap and Bolt at Foot of Rafter

with that between the
rafter. The forces acting on
are the only ones that
sideration. These are:

SINGLE SHEAR = $S/2$ =
sion in the strap on one side

BEARING PRESSURE pe
length = S/B, where B is
of the tie-beam in inches

BEARING PRESSURE pe
length between strap and bo

To find the value of S,
polygon is drawn as shown at
in Fig. 32. T is drawn para
rafter and with a length,
venient scale, equal to the thrust. From the end a an indefinite line
parallel to the axis of the strap, and from b another line perpendicul
SEAT of the rafter. These intersect at c, so that ac, measured by
scale used in laying off T, is the magnitude of the force S in the stra
rafter rests on top of the beam, bc is vertical, but if the tie-beam is d
shown by the dotted line, the line from b is drawn perpendicular to th
of the notch, making the intersection at c'. It is seen that notchin
beam in this way increases the stress in the strap.

Example 15. It is required to determine the size of a strap and
to hold the rafter without notching into the tie-beam of a long-leaf y
king-post truss. The rafter is 6 by 6 in, is inclined at an angle of
under a compressive stress of 18 000 lb. The tie-beam is 6 by 8 in in

Solution. Since the inclination is 45°, a consideration of the forc
in Fig. 32 shows ab equal to ac, so that

in, a sufficient thickness if the strap were strong enough to develop a
pressure over the rafter. It is not good practice, however, to use such
erial, because of the danger of loss of strength due to corrosion. No
s than ⅜ in thick should be used in such places.
earing-pressure per inch, between the strap and the bolt, for a ⅜-in
000/⅜ = 24 000 lb
olt, then, must take a single shear of 9 000 lb, a bearing pressure of
against the wood for each inch of length, and a bearing of 24 000 lb
of length against the strap. From Table IX a 1⅛-in steel bolt is
to resist the shear, from Table V a 2-in bolt is large enough to resist
ng from the strap, and from Table VII a 2¼-in bolt is found necessary
the 3 000-lb bearing from the wood per inch of length of bolt. This
e 2¼-in bolt satisfactory for the joint.
ressure from the bolt to the wood, however, is not parallel with the
t inclined at 45°. The allowable pressure against wood across the
about one-fourth of that with the grain. According to the formula
Chapter XXVIII, page 1138, the allowable pressure per square inch for
is 612 lb instead of the 1 400 per sq in allowed for direct compression
grain. The reduced allowable pressure makes it necessary to use a
lt, say a 5-in bolt, which would be impracticable, for it would almost cut
eam in two. It thus appears that this form of joint is not good design
ss of this span. For shorter
he joint may be made in
ce with the requirements
It has the advantage of not
g any projections below the

V. Bolts in Tension to
e Foot of a Rafter. In the
own in Fig. 33 the bolt is
to DIRECT TENSION only.
unt of the tension *S* is found
construction explained in
The rafter may be let into
eam or rest on top of it, the
in the bolt being less in
r case; but it is easier to erect the truss if the rafter is NOTCHED INTO
M from 1¼ to 1½ in for ordinary spans and loads, to hold it while the
pieces are fitted. After this is done, the holes may
be bored exactly where required.

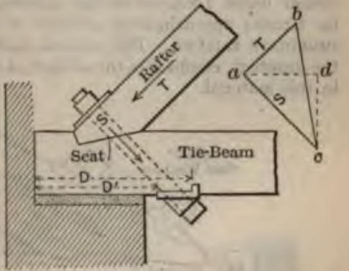
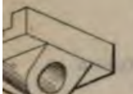


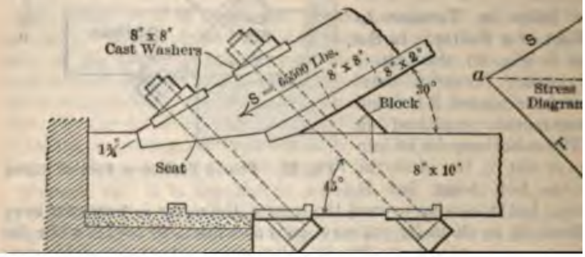
Fig. 33. Bolt in Tension at Foot of Rafter


Special Washer

Whenever *S* exceeds about 10 000 lb for trusses
made of timber for which the highest bearing stresses
are allowed, a CAST PLATE, as shown in Fig. 34 and
made to fit the inclination of the bolt, should be let
into the tie-beam at the head of the bolt to distribute
the pressure. The diameter of the hole for the bolt
e ⅛ in larger than the diameter of the bolt. The distance *D* must be
fficient to provide for the horizontal component of *S*, at the allowed
stress of the material for shear with the grain.
orizontal component is found by drawing a vertical line from *c* and a
al line from *a* and measuring *ad* to the scale of the diagram. For
his force must be less than the *product of the distance D, the width of*
n and *the allowed shearing-stress given in Table I, page 412.*

found as directed above, scales 19 000 lb. The width of the tie-be
which at the allowed shearing-stress, 150 lb per sq in, gives 900 lb a
that must be cared for by each inch of *D*. 19 000 lb divided by 900
the required distance *D*. (See, also, Chapter XXVIII, Joints i
Trusses.)

The compression against the grain on the end of the cast-iron w
also be investigated. 19 000 lb divided by the width, 6 in, gives 3
must be resisted per inch of width of beam. At 1 400 lb per sq in, a
able working stress, this makes it necessary to set the casting 2¼
lower side of the beam, which exceeds the depth usual in ordinar
Some tests made at the Massachusetts Institute of Technology on la
and reported in 1897, indicated that for a TEST CARRIED TO RUPTURE
prescribed for usual designs might safely be more than doubled.
timber under LONG-CONTINUED LOADING indicate that rupture fin
for stresses approximating one-half of those developed in TESTS C
IMMEDIATE FAILURE. This, and the fact that decay may affect the
the members, emphasizes the wisdom of using CONSERVATIVE WORKIN
in this material.



he joint even when the truss is fully loaded.

tically the washers on the top surface of the rafter should be deter- the allowable working stress in compression across the fibers. This eaf pine is taken at 350 lb per sq in (Table VI, page 454). The area, 48 250/350, or 138 sq in. This requires a washer 11¾ in square. 8-in washer used, assumes a pressure of 755 lb per sq in, but as the tests rest Service of the United States Department of Agriculture give 3 480 in as the elastic limit for long-leaf yellow pine, it is very likely that uld be no signs of injury at this point, other than a SLIGHT INDENTA- en the truss is fully loaded.

CHAPTER XIII

BEARING-PLATES AND BASES FOR COLUMNS, AND GIRDERS. BRACKETS ON CAST-IRO COLUMNS*

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1. Bearing-Plates and Bases

The Purpose of Bearing-Plates or Bases. When a heavily l
umn, beam or girder is supported on a masonry wall or pier, a BEARIN
BASE of suitable dimensions must be used to distribute the load s
pressure will not exceed the safe BEARING STRENGTH of the masonry

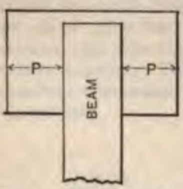

Fig. 1. Simple Bearing-plate

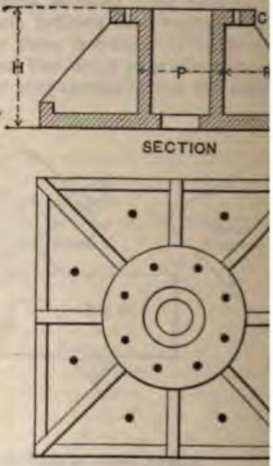

Fig. 3. Ribbed Cast-iron Pla

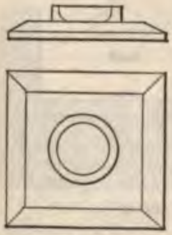

Fig. 2. Beveled Cast-iron Plate with Pin

The BEARING-PLATE is designed to be stiff enough to distribute the
under it uniformly, and its area is determined by dividing the load
the allowable pressure per unit of area (Table II).

Simple Bearing-Plates. Fig. 1 shows a SIMPLE BEARING-PLATE
beam. It may be a steel or cast-iron rectangular plate of sufficient
to prevent its bending at the edge of the beam from the pressu

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* See, also, Chapter XIV, Subdivisions 8 to 11.

elow. For anchors for steel beams on bearing-plates, see Chapter 619.

on Plates with Pin. Fig. 2 is a cast-iron PLATE WITH A DOWEL-PIN de the shell of a cast-iron column, or into a recess cut in the bottom len one. The pin holds the base-plate in position.

ron Ribbed Bases. Fig. 3 is a cast-iron RIBBED BASE for a large l cast-iron column, capable of supporting a load heavy enough to late similar to the one shown in Fig. 2, at the edges of the column, plate were made unduly thick.

I. Allowable Bearing Pressure on Different Kinds of Masonry

Kind of masonry	Allowable pressures	
	Lb per sq in	Tons per sq ft
From the building laws of New York and Philadelphia		
n lime mortar	112	8
n lime-and-cement mortar	162	12
n cement mortar	208	15
masonry, in cement mortar	140	10
e	208	15
mmended by a committee of Chicago architects and engineers in 1907		
, in lime mortar	60	4.32
in Portland-cement mortar	100	7.2
rubble, in lime mortar	120	8.6
in cement mortar	200	14.4
limestone, in cement mortar	400	28.8
granite, in cement mortar	600	43.2
e, 1 : 2 : 4, hand-mixed	350	25.2
machine-mixed	400	28.8

ases of the Steel Cores of Composite Columns used in reinforced-construction have areas sufficient to distribute the loads of the columns concrete in the footings at the allowable working stress of the concrete, usually 500 lb per sq in. (See, also, page 474, Figs. 14 and 15.)

e 1. The basement-columns of a warehouse are designed for a load o lb each. It is required to determine the size of the base-plates to he concrete foundations.

n. At an allowable pressure of 208 lb per sq in, the required area is 208 or 1 020 sq in, or about 32½ in square. The plan and section of plate is shown in Fig. 3.

of Base-Plates. For small columns and wooden posts with light AIN FLAT PLATES of cast iron or steel are generally used. The cast-iron ay have a raised ring or cross to fit inside a hollow metal column, or from 1½ to 2 in in height for a wooden *one. If the plate is very thick edges may be beveled to save weight, as shown in Fig. 2, but no part d be less than about ¾ in thick.*

Table II. Allowable Loads on Standard, Steel Bearing-Plates on Wal

Bearing on wall, in	Size of plate, in	Safe bearing value of plate in pounds		
		Bricks laid in mortar of		
		Lime, 112, lb per sq in	Lime and cement, 162 lb per sq in	Cement, lb per s
6	6×6	4 070	5 800	7 50
	6×8	5 400	7 800	10 00
	6×10	6 700	9 700	12 50
8	8×8	7 200	10 200	13 30
	8×10	9 000	12 100	16 60
	8×12	10 700	15 500	20 00
10	10×10	11 200	16 200	20 80
	10×12	13 450	19 500	25 20
	10×14	15 700	22 700	27 90
12	12×12	16 150	23 300	30 00
	12×14	18 800	27 400	35 00
	12×16	22 000	31 200	40 10
	12×18	24 200	34 500	45 00
14	14×14	22 100	31 800	40 80
	14×16	25 000	36 300	46 60
	14×18	28 200	40 800	52 40
	14×20	31 400	45 400	58 20
16	16×16	28 700	41 500	53 20
	16×18	32 300	46 600	59 80
	16×20	35 800	51 900	66 70
	16×22	39 500	57 000	73 20

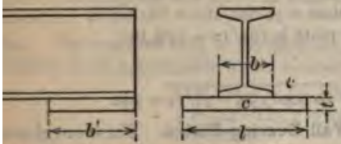
The last column applies to concrete walls also.

Ribbed Bases. If the calculated size of a bearing-plate is so large t projection beyond the edge of the column would be more than about RIBBED BASE similar to that shown (Fig. 3) for a cylindrical column is For such bases it is unnecessary to consider the transverse stresses. these bases are bolted to the columns they add greatly to the general st of the supporting members because of the greater width of such bases.

Proportions of Ribbed Bases. The HEIGHT *H* of this type of base be approximately equal to the PROJECTION *P*, and the DIAMETER *D* eq the diameter of the column. The projection *C* should be at least 3 in to the bolting of the column to the base. The THICKNESS of all parts of the c should be the same and approximately equal to the thickness of the co shell. There must be no thin webs as they result in breakage from shri stresses.

Base-Plates for Steel Columns are usually made of STEEL PLATES and s as shown on the channel-columns in Chapter XIV, Figs. 17, 18 and 19. iron bases are sometimes used for very heavy columns. If conditio favorable to the action of corrosion the cast iron is to be preferred.

'hickness of the **Bearing-Plate** is found by the formula used to e the flexure of beams. It must be determined in each case. For a ise the forces acting are shown in Fig. 5, which represents a transverse



4. Simple Bearing-plate under I-Beam

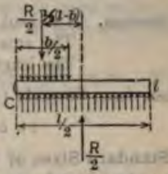


Fig. 5. Forces Acting on Half of Bearing-plate

ection through one-half the plate. The vertical section at *C*, and nd parallel with the web of the **I** beam, is taken through the center of which is the dangerous section, or section of maximum bending mo-

. 4 and 5, b' is the bearing depth on the wall;
l is the length of the plate, parallel with the wall;
b is the width of the flange of the beam;
R is the load on the bearing-plate.

ng the uniform loads by the equivalent forces at the center of gravity hese forces are represented by the longer arrows. The bending mo- he section at c is the same as the moment of the concentrated forces,

$$M = (R/2 \times l/4) - (R/2 \times b/4)$$
$$M = R/2 \times (l - b)/4$$

ual to the resisting moment at the same section c, or, at stress S,

Example 2. It is required to determine the length and thickness
iron bearing-plate under a wooden beam which is 10 in wide and
load of 24 000 lb. The plate is 8 in wide and bears that width on a b
laid up in lime mortar.

Solution. The load on the plate is 24 000/2 = 12 000 lb. From
the area of the plate is 12 000/112 = 108 sq in. Hence, if the wid
plate is 8 in, its length must be 13½ in. Then, from Formula (1)

$$t = 0.0158\sqrt{12\,000\,(13\tfrac{1}{2} - 10)/8} = 1.15 \text{ in}$$

A plate 1¼ in thick would be used.

Example 3. It is required to determine the length and thickness
bearing-plate under the end of a 24-in 80-lb **I** beam supported on a 1
wall laid up in lime-and-cement mortar and carrying a load of 60 000
width of the flange of the beam is 7 in.

Solution. The load on the plate is 60 000/2 = 30 000 lb
The area of the plate = 30 000/162 = 185 sq in
The length of the plate is 185/12 = 15½ in

Then, from Formula (2)

$$t = 0.00685\sqrt{30\,000\,(15.5 - 7)/12} = 1 \text{ in}$$

Standard Sizes of Steel, Wall Bearing-Plates. These are given
II, and are based upon ALLOWABLE PRESSURES of 112, 162 and 208 lb
These UNIT PRESSURES are based upon the ALLOWABLE PRESSURES of
York and Philadelphia building laws whic
pressed in tons per square foot. Becau
complicated formula on which the thickness
it is best to compute the thickness for each

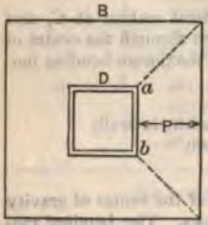


Fig. 6. Flat Bearing-plate for Column

Bearing-Plates under Columns. Th
rules already given for the proportions o
BASES similar to that shown in Fig. 3 are a
guide for detailing such bases; but in ca
FLAT PLATES are used under columns, their
must be computed according to the principle
ing bending. The stress in a FLAT PLATE
at the middle and subjected to a uniform lo
be determined by the ordinary methods of m
The approximate solution here given is
used in the design of BASE-PLATES and COLUMN-FOOTINGS. It giv
found to be safe in practice.

In Fig. 6, let B = the length of the side of the plate as determine
allowable pressure on the supporting masonry;
D = the side or diameter of the column;
$P = (B - D)/2$ = the projection of the plate;
t = the thickness of the plate;
A' = the area of the plate outside the column;
w = the allowable bearing pressure on the masonry d
load on the column.

Then in Fig. 6, the pressure on one-fourth of A', shown enclosed by t
lines in the figure, causes shearing and bending stresses in the sectio
plate along the line ab. Considering the part enclosed and taking
about the section ab, the following equation is obtained from the us

it formula. (See Chapter XV, page 557.) That is, the resisting
quals the bending moment, or

$$SI/c = \tfrac{1}{4} A'Pw$$

ctangular section at *ab*, this may be written

$$St^2D/6 = \tfrac{1}{4} A'Pw$$

$$t = \sqrt{3\,A'Pw/2\,SD}$$

omes for $S = 3\,000$

$$t = 0.0224\,\sqrt{A'Pw/D}$$

$$S = 16\,000$$

$$t = 0.0097\,\sqrt{A'Pw/D}$$

4. It is required to determine the size and thickness of a cast-iron
ate to be used under a wooden post 12 in square in cross-section and
or a load of 115 200 lb. The plate is to be set on brickwork laid in
ortar.

. The required area of the base is $115\,200/208 = 565$ sq in. $\sqrt{565}$
nd a 24-in square plate would be used.

$$A' = 576 - 144 = 432 \text{ sq in}$$

$$P = (24-12)/2 = 6 \text{ in}$$

$$D = 12 \text{ in}$$

$$w = 208 \text{ lb per sq in}$$

$$t = 0.0224\,\sqrt{432 \times 6 \times 208/12} = 4.75 \text{ in}$$

ckness may be beveled to 1½ in at the edge. The computed thickness
than is usual for such plates, some formulas having more practical
which really assume a stress of about 10 000 lb per sq in in cast iron
t.

late is made of steel

$$t = 0.0097\,\sqrt{432 \times 6 \times 208/12} = 2 \text{ in}$$

2. Bearing-Brackets on Cast-Iron Columns

ual Column-Connections for fastening beams and girders to cast-
ns are shown in Fig. 7.* The end of the beam or girder is set on
, under which is a BRACKET-SUPPORT *C*, cast on the side of the column.
gle beam, one bracket is sufficient; for wide beams or girders there
two ribs. The ends of the beams are fastened to the column by
LUGS *L*, cast on the column above the bracket. Sometimes a column
l by bolts passing through the bottom flange of the beam and through
plate. This connection greatly decreases the lateral stability of a
nd should not be used.

helf and Brackets, when loaded, are subject to SHEARING and BEND-
SES. The SHEAR at the outer surface of the column-shell is equal
l-reaction of the beam it supports. The BENDING-STRESS is due to the
n of the load on the shelf-plate at some distance from the surface of
n. It causes a tension at the top of the bracket which tends to tear
hell of the column, and causes, also, a compression at the foot of the
THICKNESS OF THE RIB must be great enough to *withstand the com-*
rom the load above; and *since the stress is variable along a section,*

---

* *See also, Figs. 5 and 7, pages 457 and 458.*

as along the line *X*, a rough approximation may be made by assum stress at the extreme edge to be twice the average stress, and by further ing that the section in the rib takes care of all the compression. This ma unnecessary to find the CENTER OF GRAVITY and the MOMENT OF INERTIA section at *X*, both of which must be known if the FLEXURE-FORMULA is This procedure, also, makes unnecessary any assumption as to the true p of the CENTER OF PRESSURE on the top surface of the bracket. With the ness of rib given in the tables there is an ample FACTOR OF SAFETY for an

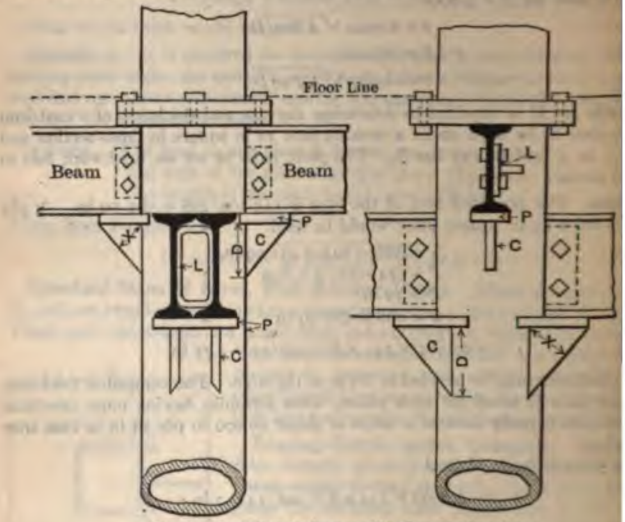


Fig. 7. Cast-iron Columns with Bearing-brackets

that may be applied through a beam. The double ribs are required whe beams are used, not for strength, but to prevent the failure of the shell ECCENTRIC LOADING.

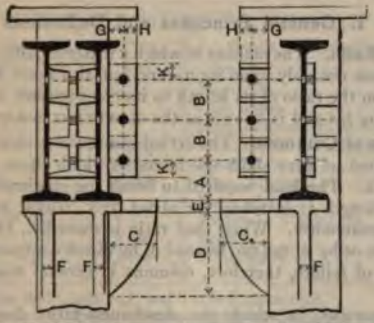
Tests of Cast-Iron Brackets. Brackets of cast-iron columns tested New York Building Department gave a SHEARING STRENGTH of 4 200 sq in on the section at the column when the load was applied at the end bracket, and an average of 8 000 lb per sq in when the load was distribute the bracket-shelf. The RANGE OF STRESS in the first case was from 2 5 600 and in the second from 4 100 to 10 900 lb per sq in. In seventeen twenty-two tests the MANNER OF FAILURE was the tearing out of a hole body of the column. It appears that when the thickness of the rib and is the same as that of the shell of the column, there is generally ample st for the support of beams and girders; but that in the case of very h loaded beams, the SHEARING and CRUSHING STRENGTH should be investi From the results of the tests mentioned, a low WORKING STRESS FOR must be assumed.

The Bevel of Brackets. If the shelf *P* (Fig. 7), on which the beam is cast SQUARE with the column, when the beam deflects, the load is brou the extreme end of the bracket, causing an increased bending-stress and connections and tending to tear a hole in the column-shell. To is the bracket-shelf should be sloped downward, away from the column, ld have a BEVEL of ⅛ in to the foot.

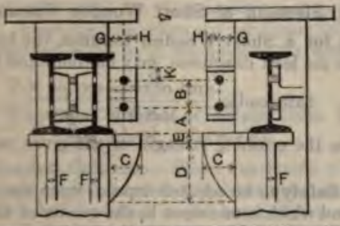
ard Connections for Cast-Iron Columns. Table III, published r in the Passiac Rolling Mill Handbook, and widely used by other turers, will be found useful when detailing cast-iron columns.

Table III. Standard Connections for Cast-Iron Columns

All dimensions are in inches



A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*K*	Thickness of lugs	Holes cored for ¾-in bolts
5	5	6	10½	1½	1½	2	1½	2	1	Holes cored for ¾-in bolts
4	5	6	10½	1½	1½	2	1½	2	1	
4	3½	5½	9½	1½	1¼	2	1½	1¾	1	
3	3	4½	7¾	1¼	1¼	2	1½	1½	1	



A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*K*	Thickness of lugs	Holes cored for ¾-in bolts
3¼	3½	4	7	1¼	1	2	1½	1½	1	Holes cored for ¾-in bolts
3	3	4	7	1	1	2	1½	1½	1	
2½	3	4	7	1	1	2	1½	1½	¾	
2¾	2½	4	7	1	1	2	1½	1¼	¾	

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1. General Principles and Definitions

Slenderness-Ratio. The manner in which a material fails under c or pressure depends not only upon its nature, but also upon its dim form, that is, upon the ratio of its length to its cross-section or diam ratio is denoted by l/r and is known as the SLENDERNESS-RATIO.

Three Classes of Columns. The actual compressive strength o must be determined on very short specimens in which there is no t bend or to buckle. The load required to break the specimen does much until the length is increased to about TEN TIMES THE DI LEAST LATERAL DIMENSION. When that ratio is exceeded, the spec to fail by BENDING or by BUCKLING instead of by direct compression. to their manner of failure, therefore, columns in general may be d three classes:

(1) Short Columns, in which the slenderness-ratio does not and which fail by direct compression.

(2) Columns in which the slenderness-ratio varies from 10 to 30 and cast iron and from 10 to 90 for steel. The failure of columns o is due partly to direct compression and partly to bending.

(3) Long Columns, in which the slenderness-ratio exceeds 30 for cast iron and 90 for steel. These columns fail wholly by bending o which causes flexural stresses of compression and tension on the co convex sides respectively.

2. Strength of Short Wooden Columns

The Safe Load for a Short Wooden Column, the length of w more than 10 times the least dimension, may be computed by the fo

$$\text{Safe load} = \frac{\text{area of cross-section} \times S}{}$$

might cause it to give way. Wooden posts supporting machinery, or uts in railway bridges, should have a factor of safety of from six to e values of S given in Table I are used.

* Average Crushing-Loads in Pounds per Square Inch, for Building Materials

faterials	Crushing-loads, lb per sq in	Materials	Crushing-loads, lb per sq in
	S		*S*
., brick, concrete		Woods (continued)	
asonry, see Chap-		Cypress	3 500
		Hemlock	4 000
Metals		Oak, white	5 000
		Pine, long-leaf yellow	5 000
	80 000	Pine, short-leaf yellow	4 000
iron	55 000	Douglas fir	4 500
led shapes	60 000	Pine, Norway	3 500
		Pine, white	4 000
with the grain†		Redwood, California	4 000
	3 500	Spruce	4 500
	4 000	Whitewood	3 000

also, Table XVI, page 647, and Table I, page 1138.

e are values for wooden columns under 15 diameters in height and are, of rage values. For crushing resistance of timbers perpendicular to the grain, I.

1. What is the safe load for a long-leaf yellow-pine column, 10 by 10 -section and 12 ft long, using a factor of safety of 5?

Area of cross-section = 100 sq in; safe load per sq in = 5 000/5 = 1000; o = 100 000 lb.

2. It is required to support a brick wall weighing 80 000 lb by a r column 11 ft long. What should be the cross-section of the column?

As previously stated, for these conditions it would be wise to use a afety of 6. Then the safe resistance per square inch of section-area = 750; 80 000/750 = 106 sq in required, about equivalent to a 10 by -section.

ength of Wooden Columns or Struts Over Ten Diameters in Length. Formulas

as for Wooden Columns. When the length of a column exceeds times its least cross-dimension it is liable to bend under the load, and reak under a less load than would break it if it were shorter and of ross-section. To deduce a formula which will make the proper allowhe length of a column has been the aim of many engineers, but their ave not always been exactly verified by actual results.

cently the formulas of Lewis Gordon and C. Shaler Smith have been ally by engineers, but the extensive series of tests made by the Governng-machine at Watertown, Mass., on full-sized columns, showed that ulas did not agree with the results there *obtained. James H. Stan*e year 1891 *plotted the values of all the tests made at the Watertown to that time on full-size columns. From the results thus obtained*

III, IV and V for the strength of round and square columns of the s used in practice. Of course other formulas must be used when req tain city building laws. For other sizes the loads can easily be the formulas. For columns having bad knots or other defects, c 10 or 12% of moisture, or which are to be exposed to the weather be eccentrically loaded, a deduction of from 10 to 25% should be m values given in the tables.

The loads for columns of other species of wood were computed by formulas of the same form as that of Formula (2):

For Douglas fir and spruce,

$$\text{Safe load per square inch} = 850 - 8.5 \times \frac{\text{length in inche}}{\text{breadth in inche}}$$

For chestnut, hemlock, short-leaf yellow pine and white pine,

$$\text{Safe load per square inch} = 750 - 7.5 \times \frac{\text{length in inche}}{\text{breadth in inche}}$$

For cedar, cypress, redwood, Norway pine and whitewood,

$$\text{Safe load per square inch} = 625 - 6 \times \frac{\text{length in inches}}{\text{breadth in inche}}$$

In these formulas the breadth is the least side of a rectangular co diameter of a round column. The round columns were computed inch, to allow for being turned out of a square column, of the nex The formulas were used only for columns exceeding 12 diameters fir and spruce, and exceeding 10 diameters for other woods.

4. Tables of Safe Loads for Wooden Columns

Tables II, III, IV and V give the safe loads in pounds for round wooden columns of different cross-sections and lengths and of diffe wood. They were computed from formulas as explained above favorable conditions of material, seasoning and position in buildin

---

* There are many formulas for the safe loads per square inch of cross-sect columns. Among those frequently used are the following:

Safe Loads in Pounds for Long-Leaf Yellow-Pine and White-Oak Columns, Round and Square

mn	Length of column in feet								
	8	10	12	14	15	16	18	20	24
....	18 200	16 800	15 360	.....	.....	.....	.....	.....	.....
....	19 590	18 760	17 550	16 500	.....	.....	.....	.....	.....
....	30 200	28 800	27 400	25 900	25 200	24 500	.....	.....	.....
....	40 300	38 400	36 500	34 600	33 600	32 600	.....	.....	.....
....	50 400	48 000	45 600	43 200	42 000	40 800	.....	.....	.....
....	38 540	37 130	35 710	34 300	33 590	32 890	.....	.....	.....
....	64 000	54 400	52 500	50 600	49 600	48 600	46 700	.....	.....
....	80 000	68 000	65 600	63 200	62 000	60 800	53 400	.....	.....
....	96 000	81 600	78 700	76 800	74 400	73 000	70 100	.....	.....
....	70 900	61 970	60 190	58 350	57 429	56 580	54 800	.....	.....
....	100 000	100 000	85 600	83 200	82 000	80 800	78 400	76 000	.....
....	120 000	120 000	102 700	99 800	98 400	97 000	94 100	91 200	.....
....	140 000	140 000	119 800	116 500	114 800	113 100	109 800	106 400	.....
....	103 900	103 900	90 912	88 730	87 690	86 550	84 160	82 290	.....
....	144 000	144 000	144 000	123 800	122 400	121 000	118 100	115 200	109 440
....	168 000	168 000	168 000	144 500	142 800	141 100	137 800	134 400	127 680
....	192 000	192 000	192 000	165 100	163 200	161 300	157 400	153 600	145 920
....	196 000	196 000	196 000	196 000	170 900	169 100	165 800	162 400	155 800
....	256 000	256 000	256 000	256 000	229 100	225 300	221 400	217 600	209 900
....	324 000	324 000	324 000	324 000	324 000	289 400	285 100	280 800	272 160
....	400 000	400 000	400 000	400 000	400 000	400 000	356 800	352 000	342 400

Safe Loads in Pounds for Douglas-Fir and Spruce Columns, Round and Square

mn	Length of column in feet								
	8	10	12	14	15	16	18	20	24
....	15 500	14 230	13 030	.....	.....	.....	.....	.....	.....
....	16 650	15 790	14 900	14 030	.....	.....	.....	.....	.....
....	25 704	24 480	23 256	22 032	21 420	20 808	.....	.....	.....
....	34 272	32 640	31 008	29 376	28 560	27 744	.....	.....	.....
....	42 840	40 800	37 760	36 720	35 700	34 680	.....	.....	.....
....	32 740	31 540	30 310	29 140	28 540	27 940	26 740	.....	.....
....	47 870	46 240	44 600	42 970	42 160	41 340	39 710	.....	.....
....	59 840	57 800	55 760	53 720	52 700	51 680	49 640	.....	.....
....	71 808	69 360	66 910	64 460	63 240	62 000	59 560	.....	.....
....	54 150	52 650	51 150	49 580	48 820	48 070	46 570	.....	.....
....	85 000	78 800	72 760	70 720	69 700	68 680	66 640	64 600	.....
....	102 000	89 760	87 300	84 860	83 640	82 400	80 000	77 500	.....
....	119 000	104 700	101 860	99 000	97 580	96 150	93 300	90 400	.....
....	88 290	79 100	77 250	75 400	74 470	73 550	71 700	69 850	66 160
....	122 400	110 160	107 700	105 260	104 040	102 800	100 360	97 920	93 000
....	142 800	128 520	125 660	122 800	121 380	119 950	117 100	114 240	108 520
....	163 200	146 880	143 600	140 350	138 770	137 080	133 800	130 560	124 030
....	166 600	166 600	149 450	146 600	145 180	143 760	140 900	138 080	132 400
....	190 400	190 400	170 800	167 500	165 900	*161 300*	*161 000*	*157 800*	*151 300*
....	217 600	217 600	*217 600*	*194 700*	*193 000*	*191 400*	*188 200*	*184 900*	*178 400*

4×6..........	13 680	12 600	11 520	.....	.....	.....	.....
5½ round.....	14 700	13 900	13 160	12 370	.....	.....	.....
6×6..........	22 680	21 600	20 520	19 440	18 900	18 360	.....
6×8..........	30 240	28 800	27 360	25 920	25 200	24 480	.....
6×10.........	37 800	36 000	34 200	32 400	31 500	30 600	.....
7½ round.....	28 900	27 850	26 780	25 720	25 190	24 660	.....
8×8..........	42 240	40 768	39 360	37 880	37 120	36 480	35 000
8×10.........	52 800	50 960	49 200	47 360	46 400	44 600	43 760
8×12.........	63 360	61 152	59 040	56 830	55 680	54 720	52 500
9½ round.....	47 960	46 440	45 160	43 740	43 100	42 400	41 120
10×10........	75 000	66 000	64 200	62 400	61 500	60 600	58 800
10×12	90 000	79 200	77 040	74 880	73 800	72 720	70 560
10×14........	105 000	92 400	89 880	87 360	86 100	84 840	82 320
11½ round....	77 925	69 820	68 160	66 490	65 770	64 833	63 170
12×12........	103 000	108 000	95 040	92 880	91 700	90 700	88 560
12×14........	126 000	126 000	110 800	108 300	107 000	105 840	103 300
12×16........	144 000	144 000	126 700	123 800	122 300	120 900	118 000
14×14........	147 000	147 000	147 000	129 300	128 100	127 000	124 400
16×16........	192 000	192 000	192 000	192 000	170 500	168 900	166 100
18×18........	243 000	243 000	243 000	243 000	243 000	217 000	213 800
20×20........	300 000	300 000	300 000	300 000	300 000	300 000	267 600

Table V. Safe Loads in Pounds for Cedar, Cypress, Redwood,
Pine and Whitewood Columns, Round and Square

Size of column in inches	Length of column in feet						
	8	10	12	14	15	16	18
4×6..........	11 520	10 550	9 800	8 700	.....	.....	.....
5½ round.....	12 350	11 730	11 180	10 490	.....	.....	.....
6×6..........	19 080	18 216	17 352	16 490	16 050	15 620	.....
6×8..........	25 440	24 290	23 140	21 980	21 400	20 830	.....
6×10.........	31 800	30 360	28 920	27 480	26 760	26 040	.....
7½ round.....	24 220	23 380	22 540	21 660	21 260	20 820	.....

5. Eccentric Loading of Wooden Columns

1 Principles. When the load on a short column or post is not axial, ıen the column supports a girder on one side only, or when the weight girder is much more than that from the others, the load is said to be c, and the distance from the point of application of the load to the axis ımn, denoted by *p*, is called the ECCENTRICITY of the load. It is evi- the stress in the column will increase with *p*, and that the total unit on the side of the column in which the on is the greatest will be greater than for axial load.

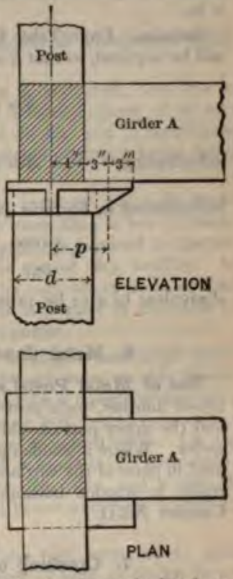

Fig. 1. Eccentric Load on Wooden Column

la for Eccentric Load. Suppose the load to be applied as shown in Fig. 1, sectional area of the required square or ar column may be computed by the fol- rmula:
ctional area of the column in square

$$A = P/S + 6\,P_1p/Sd \qquad (6)$$

A = sectional area in square inches
P = total load on column in pounds
P_1 = eccentric load in pounds
S = safe stress in pounds per square inch
p = distance from axis of column to center of bearing in inches
d = side of column parallel with girder, in inches

ming the value of *S*, the probable ratio of o the length of the column should be taken unt. Thus if it is probable that the length exceed 12 times the side, both being in inches, for oak, long-leaf yellow-pine or fir columns, or 10 times the side for other hen the value of *S* for short columns may . If the ratio will probably be greater , then the probable ratio should be roughly d and *S* computed for that ratio by the given for columns more than 10 or 12 diameters in length, as noted in paragraphs.

le 3. The lower post in Fig. 1 supports a total load on its cap-plate lb, including the reaction of 12 000 lb from girder *A*. What should be of the column if made of Douglas fir and if 12 ft in height?

n. As it is probable that the column will have to be 10 in square *S* may from Table I. With a factor of safety of 5, this is equal to 4 500/5 = r sq in. P_1 = 12 000 lb, d = 10 in and p, the distance from the axis of nn to the center of bearing of the girder = 7 in. Then from Formula (6), onal area of the column is

$$A = \frac{60\,000}{900} + \frac{6 \times 12\,000 \times 7}{900 \times 10} = 66.6 + 56 = 122.6 \text{ sq in,}$$

uivalent *to a 12 by 12-in square column. From Table III,* it may be *an 8 by 10-in column concentrically loaded will carry* almost 60 000 lb.

Hence, the eccentric load from the girder in
section of the column from 8 by 10 to 12 by

For wooden columns having a length of ov
spruce and over 10 diameters for other wo
should be found by using Formulas (3), (4)

Example 4. What size will be required fo
to carry a total load of 56 000 lb, 16 000 lb of
a girder, the distance from the center of beari
2 in.

Solution. From Table II, it is probable
will be required, so that S must be calculate

$$S = 1\,000 - 10 \times \frac{\text{leng}}{\text{brea}}$$

Substituting, $$S = 1\,000 - 10 \times \frac{168}{10}$$

Substituting in Formula (6),

$$A = \frac{56\,000}{832} + \frac{6 \times 16\,000 \times 7}{832 \times 10}$$

equivalent to a 12 by 12-in column.

6. Metal Caps and Bolsters

Use of Metal Post-Caps. Whenever
above another, each post except the top o
and the upper post should be set on the ca
girder. Where a wooden post supports a gi
used in place of the cap but modern approv
erable to wooden bolsters. Details of po
Chapter XXII.

7. Crushing of Wood Perpe

Safe Unit Stresses. The bearing of w
resting on girders, and washers on truss-ro
the quotient obtained by dividing the load
the safe unit stresses given in Table VI.

Table VI. Safe Loads for Wood P

Kind of wood	Safe loads, lb per sq in	
White oak	500	Ce
Long-leaf yellow pine	350	Sp
Douglas fir	200	He
Norway pine	200	Cy
White pine	200	Re
Short-leaf yellow pine	250	Ch

8. Cast-Iron Columns*

n Versus Steel Columns. Although steel is being used more and year for columns in buildings, it will probably never entirely supplant r buildings of moderate height. For skeleton construction, however, eight of the building exceeds twice its width, riveted steel columns, d connections with the beams and girders, are unquestionably better; e larger proportion of buildings of moderate height, cast iron will ave the preference for some time to come because it is more economi-

ges of Cast-Iron Columns. The commercial advantages which se of cast-iron columns are these:

pness. As far as the cost of production is concerned, cast iron is an steel. This consideration alone often decides in favor of its em- The raw material is easily transported as pig iron is sometimes er as ballast; so that competition with foreign countries keeps down

lability. Cast iron is the most available form of iron. An iron- quires no very elaborate plant, scarcely more than a few furnaces nolds, and moreover, no very extensive capital is required to operate uently, the product may be obtained in almost any locality. In ls, on the contrary, the machinery must be very heavy in order that ercome the enormous pressure due to the resistance of the steel in d to operate it requires a great amount of power.

diness with Which it May be Obtained. Columns and other struc- bers if made of cast iron may be obtained much more quickly than if eel. After the pattern has once been prepared, a dozen castings may lmost as quickly as one, and with but very little extra cost, except e additional raw material and the expense of remelting it. On the l, columns and girders built up of rolled sections take considerably make. Sections can be punched only one at a time, and if they do n to be of some standard length, they must be cut and fitted separately can be finally riveted together.

sical Advantages. Cast iron is one of the best materials to resist on, its ultimate compressive strength being as high as 80 000 lb per even higher. Moreover, it can be molded into almost any desired lugs, brackets and flanges may be cast upon a column all in one piece ly simplifying the cost of erection. In fact, the ease with which the girder-connections can be made is one of the chief reasons for the of cast iron. Finally, it resists fire better than steel and it corrodes Because of this, its use is advocated by many for the wall columns n structures, as these columns are particularly liable to corrode. In l and Manhattan Life Insurance Company's Buildings in New York example, the wall columns are of cast iron, whereas the interior ones l.

antages of Cast-Iron Columns. The disadvantages of cast iron for re as follows:

sical Disadvantages. Cast iron is hard and brittle and cannot be r riveted, as the blows required in driving the rivets would very likely e castings; consequently, all connections have to be made with bolts. onnection even under the most favorable conditions is not very rigid,

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* *See, also, Chapter XIII, pages 445 to 447.*

the best care and workmanship. Moreover, there are apt to be
cavities, blow-holes or honeycomb, and foreign substances, such as
sand, any of which may be on the inside of a casting, where a carefu
tion often fails to reveal them. The most critical condition, howe
due to the uneven contraction of the metal during the process of
thin parts of the casting cooling and contracting more quickly tha
ones, thereby giving rise to INITIAL STRESSES, at times of sufficien
to fracture the casting before any external loads whatever have b
upon it. In many cases this trouble is due to faulty designing or to
in handling the molds; yet, even under the most favorable conditio
difficult to secure equal radiation from the molds in all directions th
entirely free from inherent shrinkage-stresses are probably seldom

9. Design of Cast-Iron Columns

Common Forms of Cast-Iron Columns. Figs. 2, 3 and 4
common forms of cross-sections of cast-iron columns. Columns of
rectangular cross-sections are always made hollow and the diamete
made as large as possible, within reason of course; because of t

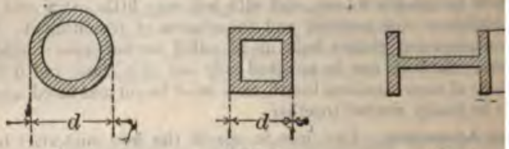
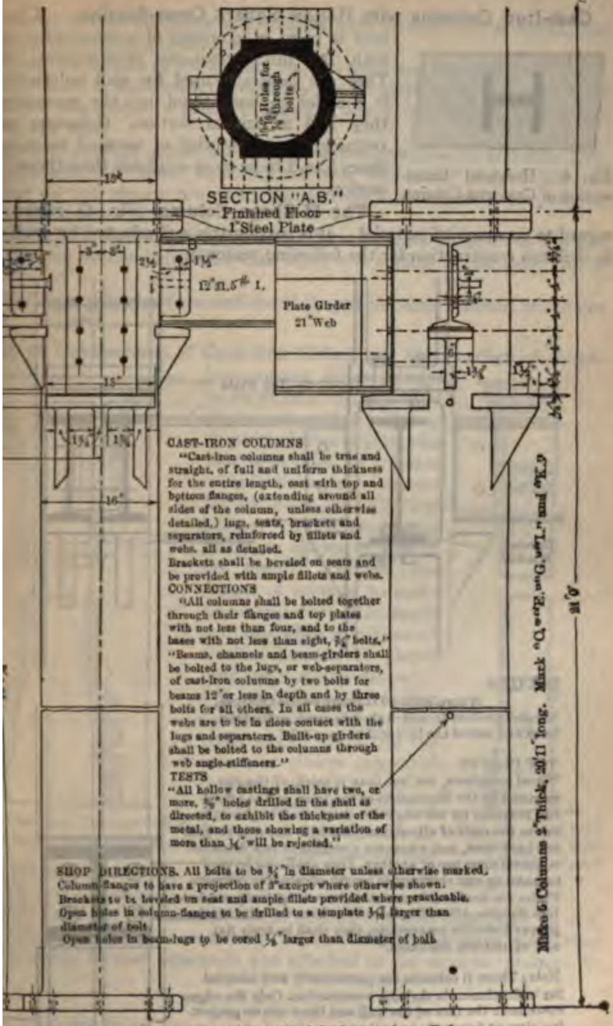


Fig. 2 Fig. 3 Fig. 4

Figs. 2, 3 and 4. Cross-sections of Cast-iron Columns

having the same area of cross-section, the one which, within certai
the greater diameter, and consequently the thinner shell, is the str
maximum thickness of shell is 1¾ or 2 in, because of the difficult
the core from shifting in columns of greater thickness; and the min


SECTION "A.B."
Finished Floor
1" Steel Plate
12" n.5# I.
Plate Girder 21" Web
10" Holes for 7/8" through bolts
15"
16"
1 1/2"
2 1/2"
1 3/4"
Make 6 Columns 2" Thick, 20'11" long. Mark "C" "E" "D" "G" "L" and "K"
21'0"

CAST-IRON COLUMNS

"Cast-iron columns shall be true and straight, of full and uniform thickness for the entire length, cast with top and bottom flanges, (extending around all sides of the column, unless otherwise detailed,) lugs, seats, brackets and separators, reinforced by fillets and webs, all as detailed.

Brackets shall be beveled on seats and be provided with ample fillets and webs.

CONNECTIONS

"All columns shall be bolted together through their flanges and top plates with not less than four, and to the bases with not less than eight, 3/4" bolts."

"Beams, channels and beam-girders shall be bolted to the lugs, or web-separators, of cast-iron columns by two bolts for beams 12" or less in depth and by three bolts for all others. In all cases the webs are to be in close contact with the lugs and separators. Built-up girders shall be bolted to the columns through web angle-stiffeners."

TESTS

"All hollow castings shall have two, or more, 5/8" holes drilled in the shell as directed, to exhibit the thickness of the metal, and those showing a variation of more than 1/4" will be rejected."

SHOP DIRECTIONS. All bolts to be 3/4" in diameter unless otherwise marked.
Column-flanges to have a projection of 3" except where otherwise shown.
Brackets to be beveled on seat and ample fillets provided where practicable.
Open holes in column-flanges to be drilled to a template 1/16" larger than diameter of bolt.
Open holes in beam-lugs to be cored 1/8" larger than diameter of bolt.

Fig. 5. Connections for Cylindrical Cast-iron Columns

ical Connections for a Cylindrical Cast-Iron Column. Fig. 5 shows tails of a cylindrical cast-iron column with typical beam and girder-con- s, dimensions and specification-notes. (See, also, *details of connections,* *ts, base-plates, etc., for cylindrical columns in Chapter XIII, Figs. 2, 3* *nd Table III of same chapter.)*

Cast-Iron Columns with Hollow-Square C

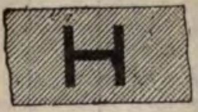
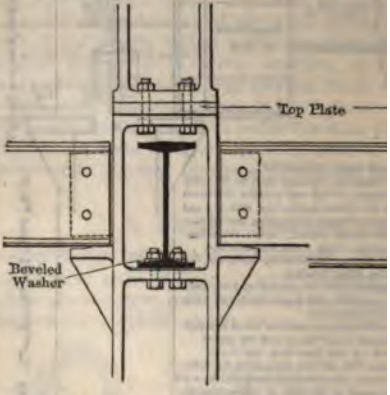


Fig. 6. H-shaped Cross-section of Cast-iron Column

next in point of econor
with the HOLLOW-SQU
They are generally us
it is easier to bond th
they had a circular
rectangular cross-sectio
times found to be more
section.

The H-Shape Colu
regard to economy of material. It is particularly
in skeleton construction for the following reasons:



BOLTING.
" Where bolts go through beveled flanges, beveled washers to match shall be used, so that the head and nut of the bolt will be parallel."

TOP PLATES.
"Steel top plates, not less than ½" thick, of the size required by the dimensions of the joint, and to afford full bearings for the angle-brackets, shall be placed between the ends of all columns cast with one side or with one back open, and whenever a column of less diameter is placed upon top of another. They shall also be used to make up any shortage in length of cast-iron columns. Plates for double columns shall be cast with top and bottom flanges. After the plates have been drilled with the proper holes for connections, they shall be truly flat and of uniform thickness."

Note: These H columns are particularly well adapted for wall columns in skeleton construction. Only the edges come near the face of the wall and there are no projecting rims or flanges to be in the way.

Fig. 7. Connections for H-shaped Cas

(1) Being entirely open, with both the interior a
any inequalities in thickness can be readily disco

red, thus obviating any necessity for drilling, and rendering the the columns much easier.

tire surface of the column may be protected by paint.

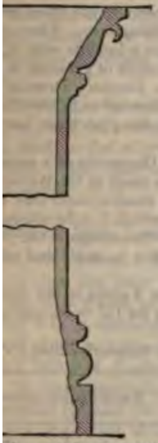
built in brick walls the masonry fills all voids, so that no open space f the column is placed as shown in Fig. 6, only its edges come near he wall.

nd brackets can be cast on such columns more readily and effectively ndrical columns, especially for wide and heavy girders, and the do not require projecting flanges, which are often in the way on olumns.

centric load may be applied to the web where its effect is less and ore evenly distributed than when it is applied to the outer rim or

connections and brackets for H-shaped cast-iron columns are shown

f Connections of Cast-Iron Columns. The bearings of a cast-iron column should always be faced true to the axis of the column, and the columns should be bolted together by four ¾-in bolts for columns 10 in in diameter or less, and by six bolts for 12-in and larger columns. Faced plates, as shown in Fig. 5, are inserted between the flanges of columns to make up for any shortage in length and also when a column of smaller diameter is placed over one of greater diameter. For convenience in erecting columns, the joint is generally placed just above the beams or girders supported by the columns.



-iron Column with e. Wrong Method

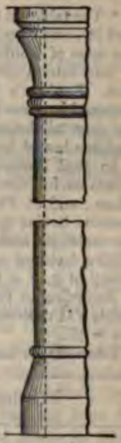


Fig. 9. Cast-iron Column with Cap and Base. Slight Projections

Projecting Caps and Bases. A column with ornamental cap and base should t as shown in Fig. 8, that is, if it is to support every bearing column, the core should extend t line from end to end. Plain molded caps and bases may be cast g. 9; but if more ornamental caps are desired, or heavy projecting should be cast separately and attached to the straight columns by

10. Strength of Cast-Iron Columns. Formulas

for Cast-Iron Columns. The ULTIMATE RESISTANCE of cast iron s generally taken at 80 000 lb per sq in, and for posts, pintels, etc., ngth is not more than six times the *diameter or breadth, it will afe to assume a* WORKING STRENGTH *of six tons per square inch of longer posts or columns, the strength is affected by the ratio of*

Ultimate strength, in pounds

$$= \text{metal-area} \times \left[80\,000 \div \left(1 + \frac{\text{sq of length in in}}{800 \times \text{sq of diam in in}} \right) \right]$$

or

$$\text{Ultimate strength, in pounds} = \frac{80\,000\,A}{1 + l^2/800\,d^2}$$

in which A is the area of the cross-section in square inches.

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* The tables in the handbook of the Cambria Steel Company are based on
and (8), and they have been adopted in the Boston building laws. They a
the form of Gordon's formula, which in turn is Rankine's formula with d,
or least lateral dimension, substituted for r, the least radius of gyration of
tion. Rankine's formula is sometimes referred to as Gordon's formula.
obtained by these formulas will be slightly in excess of those given in the Chi
law (see tabulation in this footnote), and considerably less than those per
building law of New York City, $S = 13\,000 - 30\,l/r$.

In 1898 Professor W. H. Burr made an analysis of the results of a num
ments on full-size, hollow, cylindrical cast-iron columns made at the Watert
Mass., and at Phœnixville, Pa., and by plotting the results found that a
formula having the equation $S = 30\,500 - 160\,l/d$, in which S is the ultima
the metal per square inch of column-area, represented the average of the pl
With a factor of safety of 4 this would become $S = 7\,625 - 40\,l/d$ and wi
safety of 5, $S = 6\,100 - 32\,l/d$.

According to Professor Burr's analysis the values for S given in the four
Table VII represent a factor of safety of a little over 4 for $l/d = 20$, and o
$l/d = 36$.

Formulas for finding the values of S according to the various building cod

Cylindrical columns		Rectangular colum	
Chicago	Boston	Chicago	B
10 000 l^2	10 000 l^2	10 000 l^2	

ength, in pounds

$$= \text{metal-area} \times \left[80\,000 \div 1 + \frac{\text{sq of length in in}}{266 \times \text{sq of diam in in}} \right] \qquad (9)$$

$$\text{Ultimate strength, in pounds} = \frac{80\,000\,A}{1 + l^2/266\,d^2}$$

s the area of the cross-section in square inches.
ped columns use formula (7), taking d as the least side.
LOAD is generally taken at one-eighth of the ultimate strength or
d.

Loading. Cast-iron columns should not be loaded with a heavy,
LOAD, that is, a load applied on one side of the column without a
ng load on the other side, as cast iron is unable to resist very great
esses. (See, also, eccentric loading of wooden and steel columns,
nd 485.)

bles of Safe Loads for Cast-Iron Columns. Examples

tion of Tables. As the allowable pressure PER SQUARE INCH OF
nds upon the ratio of length to diameter, without regard to actual
(that is, it would be the same for a column 6 in in diameter and 12 ft
one 8 in in diameter and 16 ft long), it is practicable to prepare a
will give the value of the terms of Formulas (7) and (8) inclosed in
all ratios of diameter to length, and thus simplify very much the
n for any particular column. Table VII has been computed by
ormulas (7) and (8) for ratios of length to diameter varying from 8
the same result will be obtained by using the values given in this
using the corresponding formula. To use this table it is only necesde the length of the column by the least thickness or diameter, both
d opposite the number in the first column of the table coming nearest
tient, find the SAFE STRENGTH PER SQUARE INCH for the column,
s multiplied by the METAL-AREA in the cross-section of the column

Tables VIII, IX and X. To still further facilitate computations, VIII, IX and X, have been prepared, which give at a glance the saf based on a factor of safety of 8, for columns of the more common si lengths. For lengths between those given in the tables sufficiently results may be obtained by interpolation. For any other factor of multiply the safe load given in the table by 8, and divide by the new of safety.

Example 6. What is the safe load for a 9-in hollow, cast-iron col square cross-section 12 ft long, the shell being 1 in thick?

Solution. From Table IX, the safe load is 129 tons. The same res be obtained by using Table VII. The ratio l/d in this case is $144/9 =$ the corresponding safe load in pounds per square inch is 8 064. The the column is 32 sq in. Hence, the safe load is 32 sq in × 8 064 lb per 258 048 lb, or 129 tons, which agrees with the safe load given in Table the same column.

Table VII. Breaking-Loads and Safe Loads in Pounds per Square for Hollow, Cylindrical and Hollow, Rectangular, Cast-Iron Colum

Calculated by Formulas (7) and (8)

Length in inches divided by external breadth or diameter	Breaking-weight in pounds per square inch		Safe loads in pounds square inch. Safety factor 8	
	Cylindrical	Rectangular	Cylindrical	Rectan
8	74 074	75 470	9 259	9
9	72 661	74 350	9 082	9
10	71 110	73 126	8 888	9
11	69 505	71 870	8 688	8
12	67 800	70 487	8 475	8
13	66 060	69 084	8 257	8
14	64 257	67 567	8 032	8
15	62 450	66 060	7 806	8
16	60 606	64 516	7 576	8
17	58 780	62 942	7 347	7
18	56 940	61 360	7 117	7
19	55 134	59 745	6 892	7
20	53 333	58 180	6 666	7
21	51 580	56 610	6 447	7
22	49 843	55 020	6 230	6
23	48 163	53 470	6 020	6
24	46 512	51 950	5 814	6
25	44 918	50 440	5 614	6
26	43 360	48 960	5 420	6
27	41 862	47 530	5 233	5
28	40 404	46 110	5 050	5
29	39 000	44 742	4 875	5
30	37 647	43 390	4 706	5
31	36 347	42 080	4 543	5
32	35 090	40 816	4 386	5
33	33 884	39 580	4 235	4
34	32 720	38 380	4 090	4
35	31 608	37 244	3 951	4
36	30 534	36 120	3 817	4

able VIII. Safe Loads in Tons of 2 000 pounds for Hollow, Cylindrical, Cast-Iron Columns with Square Ends

Based on Formula (7). Safety-factor 8

am- er, n	Thickness, in	Length of column in feet										Area of metal, sq in	Weight, lin ft
		6	8	10	12	14	16	18	20	22	24		
	¾	39	34	29	24	....	....	....	....	....	....	10.0	31.3
	⅞	45	38	32	27	....	....	....	....	....	....	11.3	35.3
	¾	46	40	35	30	26	....	....	....	....	....	11.2	35.0
	⅞	52	46	40	34	29	....	....	....	....	....	12.7	39.7
	¾	52	47	41	36	31	27	24	....	....	....	12.4	38.7
	⅞	60	53	47	41	36	31	27	....	....	....	14.1	44.0
	1	66	59	52	45	39	34	30	....	....	....	15.7	49.0
	¾	65	60	54	48	43	38	34	....	....	....	14.7	46.0
	⅞	74	68	62	55	49	43	38	....	....	....	16.8	52.6
	1	83	76	68	61	54	48	43	....	....	....	18.8	58.9
	¾	78	72	67	61	55	50	45	40	36	33	17.1	53.4
	⅞	89	83	76	70	63	57	51	46	41	37	19.6	61.2
	1	100	93	86	79	71	64	58	52	47	42	22.0	68.7
	⅞	103	98	91	85	80	71	65	59	54	49	22.3	69.8
	1	117	110	103	95	90	80	73	67	61	55	25.1	78.5
	1⅛	129	122	114	105	99	89	81	74	67	61	27.8	87.0
	⅞	118	112	106	100	93	86	79	73	67	62	25.1	78.4
	1	133	127	120	112	105	97	89	82	76	69	28.3	88.4
	1⅛	147	141	133	125	116	107	99	91	84	77	31.4	98.0
	1¼	161	154	146	136	127	118	109	100	92	84	34.4	107.4
	1	149	143	137	129	122	114	106	98	91	85	31.4	98.2
	1⅛	165	159	152	144	135	126	118	109	101	94	34.9	109.1
	1¼	182	175	167	158	148	139	129	120	111	103	38.3	119.7
	1⅜	197	190	181	171	161	151	140	130	121	112	41.6	129.9
	1⅛	184	178	171	163	154	146	137	128	120	112	38.4	120.1
	1¼	202	195	188	179	170	160	150	141	132	123	42.2	131.9
	1⅜	220	212	204	194	184	174	163	153	143	133	45.9	143.4
	1½	237	229	220	210	199	187	176	165	154	144	49.5	154.6
	1⅛	202	196	190	182	174	165	156	147	138	130	42.0	131.2
	1¼	222	216	209	200	191	181	172	162	152	143	46.1	144.2
	1⅜	242	235	227	218	208	197	187	176	166	156	50.2	156.9
	1½	261	254	245	235	224	213	201	190	179	168	54.2	169.4
	1¼	242	236	229	221	212	203	193	183	173	164	50.1	156.5
	1⅜	264	258	250	241	231	221	210	199	189	178	54.5	170.4
	1½	285	278	270	260	250	238	227	215	204	193	58.9	184.1
	1⅝	306	298	289	279	268	256	243	231	219	207	63.2	197.4
	1⅜	268	280	272	264	254	244	234	223	212	203	58.9	183.9
	1½	309	303	295	285	275	264	252	241	229	219	63.6	203.4
	1⅝	332	325	316	306	295	283	271	259	246	235	68.3	213.4
	1¾	354	346	337	327	315	302	288	276	263	251	72.8	227.6
	1½	333	327	319	310	300	290	278	267	255	243	68.3	213.5
	1⅝	358	351	343	333	322	311	299	286	273	261	73.4	229.3
	1¾	382	375	366	356	344	332	319	306	292	279	78.3	244.8
	1⅞	455	446	435	423	410	395	380	364	347	332	93.2	291.3

Table IX. Safe Loads in Tons of 2 000 Pou
Rectangular, Cast-Iron Columns,

Based on Formula (8). Safet

Size, in	Thickness, in	Length of column in fe					
		8	10	12	14	16	18
4× 6	¾	41	34	28	.....	.....	.....
4× 8	¾	51	42	35	.....	.....	.....
4× 9	¾	56	46	39	.....	.....	.....
4×10	¾	60	50	42	.....	.....	.....
4×12	¾	70	59	49	.....	.....	.....
5× 8	¾	64	55	48	41	.....	.....
	1	81	71	61	53	.....	.....
5× 9	¾	69	60	52	45	.....	.....
	1	89	78	67	58	.....	.....
5×10	¾	75	65	57	49	.....	.....
	1	96	84	73	63	.....	.....
5×12	¾	86	74	65	56	.....	.....
	1	111	97	84	72	.....	.....
6× 6	¾	63	57	51	45	40	35
	1	80	72	65	57	51	45
6× 8	¾	75	68	60	54	47	42
	1	96	87	78	69	61	54
6× 9	¾	81	73	65	58	51	45
	1	104	94	84	75	66	58
6×10	¾	87	79	70	62	55	49
	1	112	101	91	80	71	63
6×12	¾	99	90	80	71	63	55
	1	129	116	104	92	81	72
6×15	¾	117	106	95	84	74	66
	1	153	138	123	109	97	85
7× 7	¾	80	73	67	61	55	49
	1	102	94	85	78	70	63
7× 9	¾	92	85	77	70	63	57
	1	119	109	100	91	82	74
7×12	¾	111	102	93	85	77	69
	1	144	133	121	110	99	89
8× 8	¾	95	90	83	77	70	64
	1	124	115	107	99	91	83
	1¼	148	140	129	119	109	100
8×10	¾	109	103	95	87	80	73
	1	141	132	122	113	104	95
	1¼	170	161	148	137	125	115
8×12	¾	122	115	106	98	90	82
	1	158	148	138	127	116	107
	1¼	192	181	167	154	142	130

IX (Continued). Safe Loads in Tons of 2 000 Pounds for Hollow, Square and Rectangular, Cast-Iron Columns, with Square Ends

Based on Formula (8). Safety-factor 8

Thickness, in	Length of column in feet 8	10	12	14	16	18	20	24	Area of metal, sq in	Weight, lin ft
1	193	181	168	155	142	130	119	99	44.00	137.5
1¼	236	221	206	190	174	159	145	121	53.75	168.0
¾	111	106	99	93	86	80	74	63	24.75	77.3
1	144	137	129	120	112	103	96	85	32.00	100.0
1	171	162	153	143	133	123	114	97	38.00	118.8
1¼	209	198	186	174	162	149	138	118	46.25	144.5
1	207	196	185	173	161	149	138	117	46.00	143.8
1¼	254	240	226	212	197	182	168	143	56.25	175.8
1	165	158	150	142	133	125	117	101	36.00	112.5
1¼	201	193	183	172	162	152	142	123	43.75	136.7
1	184	176	167	158	148	139	129	112	40.00	125.0
1¼	224	214	204	192	181	169	158	137	48.75	152.3
1	211	202	192	181	170	160	149	129	46.00	143.8
1¼	258	247	235	222	209	195	182	158	56.25	175.8
1	220	211	200	189	178	167	155	135	48.00	150.0
1¼	270	258	245	232	218	204	190	165	58.75	183.6
1	239	228	217	205	193	181	168	146	52.00	162.5
1¼	293	280	266	251	236	221	207	179	63.75	199.2
1	257	246	234	221	208	194	181	157	56.00	175.0
1¼	316	302	287	271	255	239	223	193	68.75	214.9
1	294	281	267	252	237	222	207	180	64.00	200.0
1¼	362	346	329	311	292	274	255	221	78.75	246.1
⅞	183	177	171	164	156	149	141	126	38.90	121.7
1	207	201	193	185	177	168	159	142	44.00	137.5
1¼	253	245	236	223	216	206	195	174	53.75	168.0
1½	296	288	277	265	253	241	228	204	63.00	196.9
1	235	228	220	211	201	191	181	162	50.00	156.3
1¼	288	280	269	258	246	234	222	198	61.25	191.4
1	245	237	228	219	209	199	188	168	52.00	162.5
1	263	256	246	236	225	214	203	181	56.00	175.0
1	282	274	264	253	241	229	217	194	60.00	187.5
1	320	310	299	287	274	260	246	220	68.00	212.5
1	268	261	254	246	238	229	219	200	56.00	175.0
1	307	298	290	281	272	261	250	228	64.00	200.0
1	345	336	326	316	306	294	280	257	72.00	225.0
1	300	284	278	271	264	256	247	229	60.00	187.5
1	380	360	352	344	334	324	313	291	76.00	237.5
1	340	340	320	314	307	299	291	274	68.00	212.5
1	380	380	361	356	349	342	334	317	76.00	237.5
1	420	420	399	393	386	378	369	351	84.00	262.5

Table X. Safe Loads in Tons of 2 000 Po
Columns

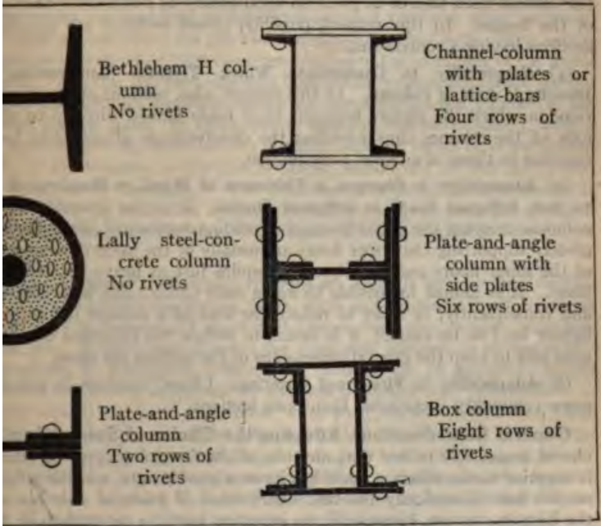
Based on Formula (7). Sa

Size, in *a* *b* *t*	Area of metal, in	Length of column 10	 12	 13
6× 6× ¾	12⅜	41	36	33
1	16	53	46	43
1¼	19⅜	64	56	52
6× 8× ¾	13⅞	46	40	37
1	18	60	52	48
1¼	21⅞	73	63	59
7× 7×1	19	69	62	58
1¼	23⅛	84	75	71
7× 9×1	21	76	68	64
1¼	25⅜	93	83	79
8× 8× ¾	16⅞	66	60	57
1	22	86	78	74
1¼	26⅞	105	95	91
8×10×1	24	93	85	81
1¼	29⅜	114	104	99
1½	34½	134	122	117
9× 9×1	25	102	94	91
1¼	30⅝	125	116	111
1½	36	147	136	130
9×10×1	26	106	98	94
1¼	31⅞	130	120	115
1½	37½	153	142	136
10×10×1	28	118	111	107
1¼	34⅜	145	136	131
1½	40½	171	160	155
1¾	46⅜	196	184	177
10×12×1	30	127	119	115
1¼	36⅞	156	146	141
1½	43½	184	172	166
1¾	49⅞	211	198	191
2	56	236	222	214
12×12×1	34	151	144	140
1¼	41⅞	186	177	172
1½	49½	220	209	203
1¾	56⅞	252	241	234
2	64	284	271	263
12×14×1¼	44⅜	197	188	183
1½	52½	233	222	216
1¾	60⅜	268	255	248
2	68	302	288	280
2¼	75⅜	335	319	310

2. Types, Forms and Connections of Steel Columns

Steel Columns, Struts, Trusses, etc. Owing to the many ad- of built-up steel columns over cast-iron columns, especially for all and to the great reduction that has taken place in the cost of steel on, built-up columns are now very extensively used in buildings of erate height; and for skeleton construction, or for buildings exceeding in height, they are certainly much to be preferred to cast-iron columns. ses, also, are now much more commonly used in buildings than in ars, so that the architect must have at hand data for designing them mputing their strength. In the following pages the author has en- to cover the subject of columns and struts quite completely, to furnish as will enable the designer to decide upon the shape of column or best to use, and also to determine the sizes and sections of such col- n the least labor.

and Forms of Steel Columns. The following are cross-sections ority of steel columns in general use, arranged in the order of their sim- construction, that is, the number of rows of rivets they require:



Bethlehem H column
No rivets

Channel-column with plates or lattice-bars
Four rows of rivets

Lally steel-concrete column
No rivets

Plate-and-angle column with side plates
Six rows of rivets

Plate-and-angle column
Two rows of rivets

Box column
Eight rows of rivets

lerations Governing the Selection of Steel Columns. There are tions other than simplicity of construction which sometimes govern ion of a column. Some of the most important of these are explained lowing paragraphs:

st and Availability of Material. I beams, channels, plates and angles most common commercial sections. They are easily rolled and are ured by all of the large mills. They are reasonable in price and may ed promptly in large numbers in any locality *where a steel building is* be erected. Pat*ented sections, or the product of one mill, do not,* as *lfill these conditions.*

possible, the rivets easy of access and as few in number as is cons
strength.

(3) Simplicity of Connections Between Column and Supported
This is quite an important consideration in the design of a large b
sometimes governs the choice of the section to be used. Where the
beams to a column, on opposite sides, and all of the same height, a s
connection can be made with almost any section; but where the
spaced irregularly, both in regard to position in plan and to height,
eccentric loads must be provided for, it is very important that the
the column itself affords as great an opportunity as possible for the c
of the beams. In this respect, possibly, closed sections are inferi
sections having a central web.

(4) Adaptability to Connections Which Transfer Compressive
Directly to Axis of Column. In this respect, also, sections of an ope
tion, in which the girders transmit their loads almost directly to t
axis of the column, thus avoiding the disadvantage of eccentric l
superior to those of a closed construction.

(5) Adaptability to Changes in Thickness of Metal in Members of
to Suit Different Loads in Different Stories. It is not desirable to
columns carrying the upper floors of a building very small, since the
girders supporting the upper floors are usually of the same dimensio
of the lower floors and consequently require just as heavy and secu
tions. It is almost impossible to make such connections with smal
and consequently, in order to reduce the area of a column in propo
lighter load to be carried, it is better to reduce the thickness of th
used and to keep the general dimensions of the section the same.

(6) Adaptability to Fire-Proof Covering. Closed sections in gene
more compactly fireproofed than open sections.

General Considerations Affecting the Choice of Steel Colum
almost impossible to say that any one of the foregoing types of ste
is superior to the others. Each has its own good points, and the col
section has theoretically the best distribution of material may not

th milled ends, and splice-plates are riveted to the sides or flanges the illustrations of typical steel-column details, Figs. 17 and 18. possible in these pages to include the subject of column-connections but a general way, the only attempt that has been made in this to illustrate common forms of connections that have been used with nds of columns. These will be found in the description of columns wing pages.

of Rivets Required. No general rule can be given for the number d size of the brackets required for column-connections, as the loads orted vary in different buildings and in different parts of the same The number of rivets required in each connection must therefore ned by the rules given in Chapter XII for designing riveted joints. s for single beams, however, will generally require the same number s are given for beam-connections (Chapter XV, page 617). The tress for rivets in column-connections is generally taken at 10 000 lb r single shear and 18 000 or 20 000 lb per sq in for bearing. (See nd III, pages 418 and 419, Chapter XII.)

of Rivets. Steel columns fail either by deflecting bodily out of a e or by the buckling of the metal between rivets or other points of Both actions may take place at the same time, but if the latter occurs y be an indication that the rivet-spacing or the thickness of the metal nt. The rule has been deduced from actual experiments upon riveted at the distance between centers of rivets should not exceed, in the ss, sixteen times the thickness of metal of the parts joined, with a spacing of 6 in, and that the distance between rivets or other points at right-angles to the line of stress, should not exceed thirty-two hickness of the metal. The usual practice in designing columns is e rivets the minimum distance on centers at both ends, for a length ice the least dimension of the column, with the maximum spacing of n.

pe Columns.* Steel-pipe columns are used for interior construction ms and girders supporting floors, walls and chimneys in all classes of uch as tenements and apartment-houses, factories, garages, churches, , etc. A particular demand for steel-pipe columns is at the angles ndows in mercantile buildings. In buildings of moderate height ists are usually supported by the side walls and the columns have to y a relatively light wall above. For such places wrought-steel pipes vantageously used for the columns. They may be used, also, for the pporting the roof of one-story buildings. In the Borough of Brook- York City, pipe-columns are calculated according to the formula $- 80\,l/r$, in which S, l and r have values as explained below for the City formula. If the columns are filled with concrete, the area of ction of the concrete is multiplied by 500 and the product added to pported by the pipe. (See, also, paragraph on Lally Columns, page s formula gives a factor of safety of four. The New York Bureau of ises the formula, $S = 15\,200 - 58\,l/r$, in which S is the permissible tress, l the length in inches and r the radius of gyration of the cross- he pipe. This gives a carrying capacity about 10% greater than that oklyn formula. In Philadelphia, pipe-columns are allowed to carry more than is allowed in Brooklyn. Where pipe-columns are filled

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aluable data relating *to steel-pipe columns was furnished the Editor-in-chief terson and J. A. McCullough of the National Tube Company, Pittsburgh, Pa.*

with concrete the cast cap and base are secured to the pipe in each ca
crete which is reinforced internally by a pipe of smaller diamete
these steel-pipe columns filled with concrete are used, care should

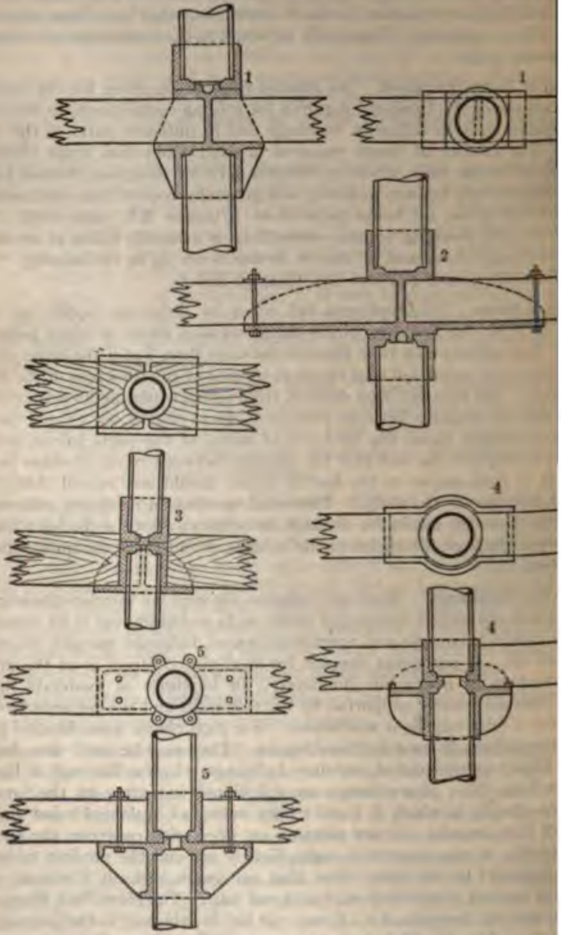


Fig. 10. Connections, Caps and Bases for Steel-pipe Columns

that the pipes are entirely filled, and that there are no air-spaces in the
These concrete-filled columns, sometimes reinforced with smaller pip
a large carrying capacity. Pipe-columns may have their supporti

ed in many cases by concrete filling. (See, also, paragraph on Lally age 477.) One type of steel post-cap used in connection with pipe-carry wooden girders is shown in Figs. 62 and 63 of Chapter XXII. nany other forms of cast and wrought caps for pipe-columns. The

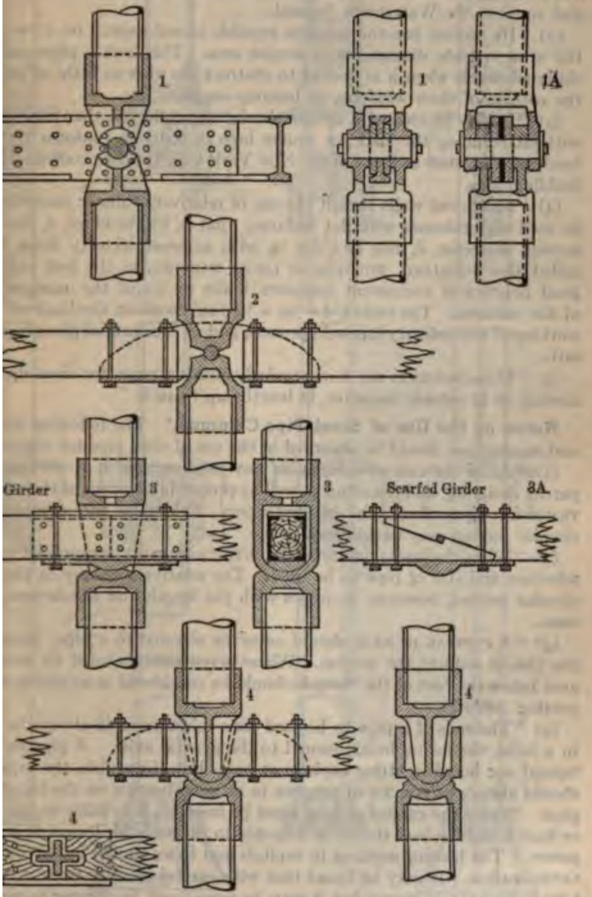


Fig. 11. Connections, Caps and Bases for Steel-pipe Columns

proper caps and bases is the most difficult part of adapting tubular practical problems in building-construction. Figs. 10 and 11 show rms of steel-pipe column-connections, caps and bases sufficiently sug- enable a designer to properly develop their details.

the corners of show-windows, in balcony-supports, etc.

(3) "It may be used with greater slenderness-ratio, l/r, than any without reducing the load per square inch in order to conform t loading-rules, such as those of the New York City building code and building code.

(4) "Its curved walls permit the use of relatively thinner materi be used with columns with flat surfaces; that is, its thickness, t, div outside diameter, d, may be $t/d = 1/80$ with as great security from called also BUCKLING, BULGING or LOCAL FAILURE, as the box col good practice of competent engineers limits to $1/30$ of the unsupp of flat surfaces. The ratio $t/d = 1/80 = 1/4''/20''$ is about the limit of working of the ordinary lap-weld process, and all commercial pipes ha ratio.

(5) "Manufacturers are now regularly making pipes for sizes up cluding 16 in outside diameter, in lengths up to 40 ft."

Notes on the Use of Steel-Pipe Columns.* The following g and suggestions should be observed in the use of steel pipe for colum

(1) "As in the case of columns of any construction, it is obvious petent designing and detailing as well as proper fabrication of the TIONS for pipe-columns be insisted upon. Otherwise the advant circular section may be nullified.

(2) "When the loading must be ECCENTRIC care must be exercised i selection and size of pipe to be used. The relative economy in th circular section, however, increases with the length and slendernes umn.

(3) "A CAPITAL or BASE should never be screwed to a pipe, bec the thread reduces the section. Where screw-threads must be us area below the root of the threads should be considered as available porting power.

(4) "The ends of a pipe to be used for a column should always b in a lathe, the facing being normal to the general axis. A pipe s turned nor bored in fitting capitals or bases but, if possible, the ca should always be FORCED or SHRUNK to an even bearing on the fac

tted usually carry heavier loads before failure. This is increasingly /r decreases. Any form of end-connection of column that may re from a failing floor may endanger the whole structure.

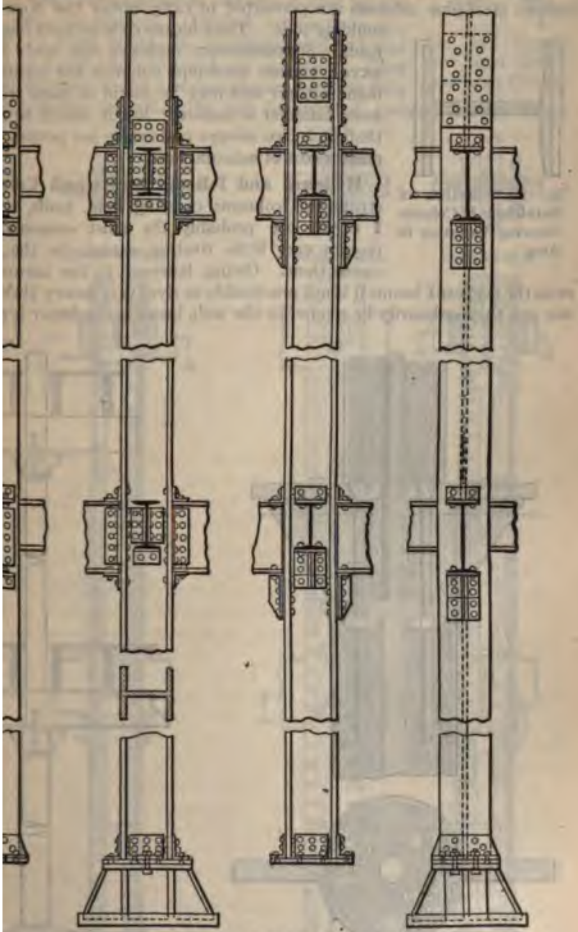
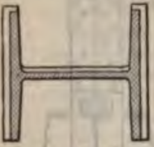


Fig. 12. Connections for Bethlehem H Columns

olumns should have sufficient STIFFNESS to safely withstand the cting forces to which they may be exposed. This usually involves s of eccentricity as well as of flexure due to transverse load. desirable to adhere *always to the trade sizes of pipe known* as *TANDARD, EXTRA STRONG, DOUBLE-EXTRA STRONG, CASING, BOILER-*

TUBES, etc., and avoid special production which usually entails delays an
prices."

(8) Tables XII and XIII give the safe loads which STANDARD and
STRONG steel-pipe columns are permitted to carry under the New Yo
building code. The Chicago code permits slightly
loads. Supplementary tables of safe loads for
EXTRA STRONG steel-pipe columns are furnished
manufacturer and may be useful in cases where
mum diameter is required; but it should be
that such pipe always costs more per pound, ow
greater cost of manufacture.



ig. 13. Section of Bethlehem H Column Showing Variation in Area

H-Beam and I-Beam Struts and Column
struts and columns carrying light loads, H BEA
I BEAMS are probably the most economical,
require very little riveting except for the spli
connections. Owing, however, to the narrow fla
even the deepest I beams it is not practicable to rivet very heavy girders to
nor can they ordinarily be riveted to the web, because the latter is ge

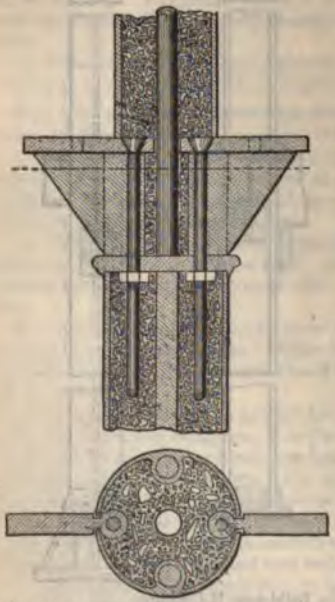


Fig. 14. Concrete-filled Lally Steel Column

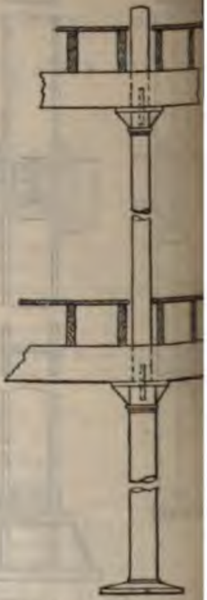


Fig. 15. Lally Column
Connections

thin that too many rivets will be required for the connection. Table
a table of safe loads for the Carnegie steel H BEAMS or I BEAMS
columns.

Columns. As far as shop-work is concerned the BETHLEHEM just as economical as the ordinary **H**-beam or ns as they, also, are rolled and not built up The only fabrication required is that for the and connections. Typical connections are . 12 from which the simplicity of detail and of fabrication required are apparent. They , superior to the **I**-beam columns because they r flange for attaching the beams and girders, besides being

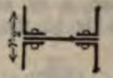
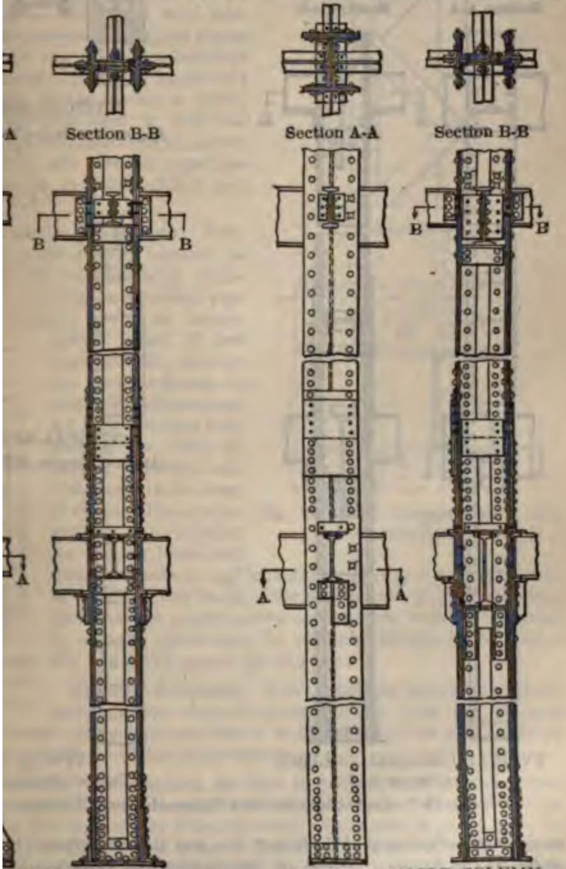


Fig. 16. Section of Steel Plate-and-angle Column

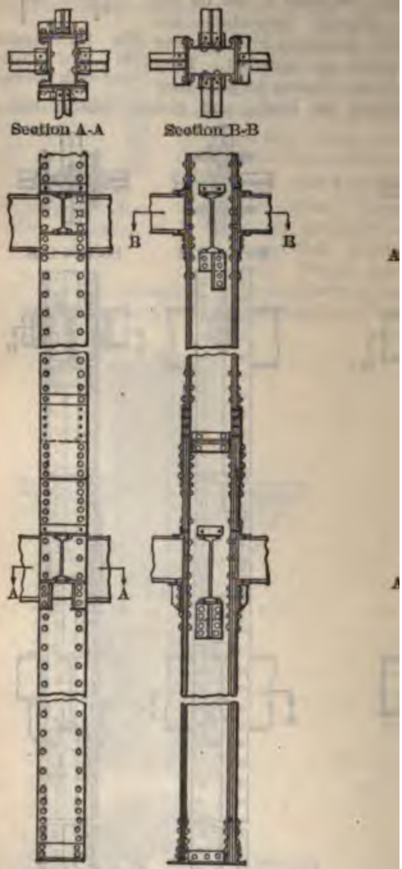


ANGLE-COLUMN TYPICAL ANGLE-COLUMN
ng on masonry Bearing on steel

Fig. 17.* Connections for Steel Plate-*and-angle Columns*

et *Companion, 1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

more economical of cross-section. Bethlehem
sizes, 8, 10, 12 and 14 in in width, but by
in Fig. 13, the section-area of each width can b



TYPICAL CHANNEL-COLUMN
Bearing on steel

Fig. 18.* Connections for Steel Plate-and

section-areas of columns of the largest size may al
plates to the flanges. Tables of DIMENSIONS
rolled steel columns and of the SAFE LOADS they
XVIII to XXI. Although these columns have

---

* From Pocket Companion, 1915 Edition, Carnegie

s not until the establishment in 1908 of the larger improved mills em, Pa., that these sections became available for use in this country. radually superseding plate-and-angle and box columns, particularly e smaller sizes.

lumns. LALLY COLUMNS (see, also, paragraphs on Steel-Pipe Col- 469) are patented columns made with a circular steel shell, as shown and filled with a concrete com- nd, cement and blue trap-rock, ghly compressed. The larger ve, in addition, a steel rein- which makes a light, but strong They are in many buildings nasonry piers for supporting ause of the saving in space, xtensively used in mill-con- Typical connections are shown The safe carrying capacities given in Tables XXII and ge 516.

d-Angle Columns. Four a plate riveted together as Fig. 16 are now being exten- in building-construction, par- r columns having an unsupported length of less than 90 radii; also for the outer columns in steel mill-buildings, and for light columns supporting the roofs of railway stations, etc. Columns with this form of cross-section are especially convenient for making beam and girder-connections and for splicing, and are also well adapted to resist eccentric loads. The width of the plate is generally such that the LEAST RADIUS OF GYRATION is in the direction r_2, and this radius may be obtained directly from Tables XV and XVII, pages 370 and 372.

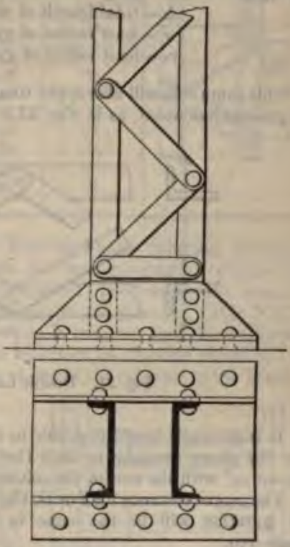
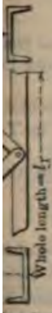


Fig. 19. Steel Channel-column with Lattice-bars



pacing -bars in lumns

Channel-Columns. Typical COLUMN-DETAILS for plate-and-angle and channel-columns, taken from the Carnegie npanion, 1915 edition, are shown in Figs. 17 and 18 and represent ctice in office-building construction.

Columns. Two channels, set back to back, at such a distance that gyration will be equal about both axes, and connected by lattice-bars, Fig. 19, make a very desirable column for moderate loads, as in the s, or in buildings of three or four stories in height. For greater cover-plates may be riveted to the flanges in place of the lattice- columns are very satisfactory, especially for making connections.

Latticing of Channels and Angles. *When channels are con- ttice-work, as in Fig. 20, in order that there may not be a tendency*

in the channels to bend between the points o
made equal to the total length of the strut mu
tion of a single channel, and the product divic
for the whole section; or,

$$l = rl_1/r_1$$

in which l = length between points of
l_1 = total length of strut;
r = least radius of gyration fo
r_1 = least radius of gyration fo

This same rule will also apply to angles, alt
is generally doubled, as in Fig. 21.

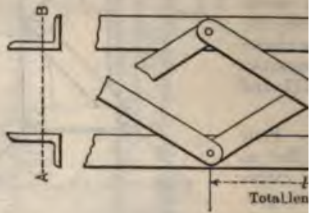


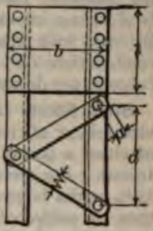
Fig. 21. Double Lattice-bars

It is generally found desirable to make the
by the above formula, or such that the inclir
about 45° with the axis of the column or strut.
The proper distance for d or D, Fig. 20, for a
of gyration will be the same in both dire
page 359.
The following tabulations are taken from
Steel Company, 1915 edition.

Sizes of Lattice-Bars to be Used with

Depth of channels	Dimensions of lattice-bars		Weight of lattice-bars per foot	C ho
	w	Thickness		
in	in	in	lb	
6	1½	¼	1.28	
7	1¾	¼	1.49	
8	2	5/16	2.12	
9	2	5/16	2.12	
10	2	⅜	2.55	
12	2¼	⅜	2.87	
15	2½	⅜	3.19	

of Stay-Plates to be Used with Latticed Channel-Columns

size of stay-plates at ds of columns		Weight of minimum stay-plates	Diameter of rivets	
Thickness	*l*			
in	in	lb	in	
¼	7½	4.38	⅝	
¼	10	6.55	⅝	
5/16	9	8.37	¾	
5/16	12	11.95	¾	
⅜	12	15.62	¾	
⅜	15	22.73	¾	
⅜	15	25.90	¾	

-Angle and Box Columns. Plate-and-angle columns, as shown equiring but two rows of rivets are very economical columns for moderate height, as they afford excellent opportunities for connect- ns and girders. Tables of SAFE LOADS are given in Table XXIV er. When a more compact section is required than that afforded sizes, the section-area may be increased by riveting plates to the wn in Fig. 22 which is a section of one of the columns in the Munic-

vy Plate- One-web

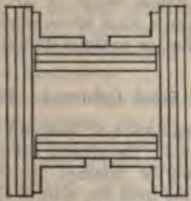


Fig. 23. Heavy Plate-and-angle Two-web Column

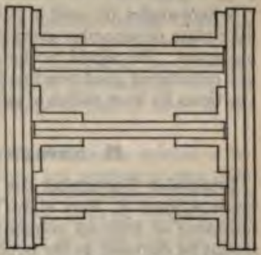


Fig. 24. Heavy Plate-and-angle Three-web Column

, New York City. This, however, greatly increases the expense work, and it is therefore usually more economical to substitute columns, or channel or box columns. For high buildings or heavy the required sectional areas of columns are greater than can be using channel-columns or Bethlehem columns without flange-plates, s made of plates and angles, as shown in Fig. 23, which is one of the he Bankers' Trust Company Building, New York City, will prob- to be more satisfactory. The thickness and number of web-plates lates can be varied with the load to be supported. Ordinary con- BOX COLUMNS are the same as those for CHANNEL-COLUMNS, shown For the tallest buildings and heaviest loads box *columns with* as shown in Fig. 24 are the *best. They are used in the highest* cted, *such as the Masonic Temple in Chicago, and the Bankers'*

Steel Struts in Trusses. These are generally made of a p
channels, or of channels and plates for heavy trusses with pin-con
of either a pair of light channels or a pair of angles with uneven
trusses. For roof-trusses having a span not exceeding 80 ft, a pai
¾-in angles is generally sufficient for any of the compression-me
they are subjected to TRANSVERSE STRESS; and the minor struts
made of a pair of 3½ by 2½ by ¼-in angles. The angles are plac
¾ in apart to permit the filler-plates used at the joints to go b
For compression-members subject to transverse stress a pair of
erally offers the best section. If necessary the channels can be
plates at the top and bottom. A pair of angles, with a deep web-
between, is often used for the principles of Fink trusses where the
to a slight transverse stress. (See, also, Fig. 6, page 1146.) F
compressive stresses and for short members a single angle is someti
this is not considered good practice, as it causes eccentric loading
plates at the truss-joints. A pair of small angles, or some other
with a symmetrical cross-section should always be used for truss-

Where angles are used in pairs they should be connected by a ri
filler-plate or separator every two feet in length, to prevent them
ing apart. In regard to the maximum length of steel struts in
not considered good practice to use a strut whose unsupported le
150 times its least radius of gyration, or 50 times its least width

13. Strength of Steel Columns. Formulas

Principles Governing the Resistance of Built-up Steel Col
fessor William H. Burr states * that "the general principles whic
resistance of built-up columns may be summed up as follows:
should be disposed as far as possible from the neutral axis of the
thereby increasing the radius of gyration, r; there should be no in
stress; the individual portions of the column should be so firmly se
other that no relative motion can take place, in order that the col
as a whole, thus maintaining the original value of r." The exper

e results differ but little from those obtained by the older formula.

as Compared. Which one, of all the formulas in use, should be n calculating the safe load for columns is an open question, but the er careful deliberation, has decided to recommend RANKINE'S FORhe following reasons. In the first place it is safe and conservative rs at all, it is on the side of safety; and in the second place it has a ication, as the values assigned to the arbitrary constants have been ally agreed upon, whereas there is a greater variety in the values of nts employed in the STRAIGHT-LINE FORMULA. Of course one is not ose when city laws compel the use of certain formulas. No tables of s for columns, satisfying the requirements of all cities, could be com- author has accordingly thought it best to insert the various tables ADS for different forms of columns as computed in the very latest although not necessarily based upon RANKINE'S FORMULA, and to le **XI**, specially computed and giving the comparative SAFE LOADS IN R SQUARE INCH OF METAL-AREA for columns, as determined by seven ormulas. (See pages 493 to 495.)

as Used in Building Codes. RANKINE'S FORMULA (called GORDON'S n many codes) is specified in the building codes of the following cities: ia, Baltimore, Boston and Milwaukee, and is used in the Cambria The STRAIGHT-LINE FORMULA is specified in the building codes of City, Chicago, Minneapolis and Washington, and is used in the nd Bethlehem handbooks.

as Used in Practice. The following formulas, in the opinion of the present the best current practice. They are FORMULAS FOR SAFE n pounds per square inch of cross-section, on steel columns and struts. rmulas l is the LENGTH of the column in inches and r the LEAST RADIUS ON of the cross-section. (See, also, Chapter X, pages 333, 344, etc.) E LOAD, P, for any column is equal to S, obtained by one of the follow- las, multiplied by the SECTION-AREA of the column in square inches;

$$P = AS \tag{10}$$

formula, used in the Cambria handbook, is

$$S = \frac{12\,500}{1 + l^2/36\,000\,r^2} \tag{11}$$

la recommended by Professor Burr is

$$S = 10\,000 - 40\,l/r \tag{12}$$

la used by the American Bridge Company and Carnegie's Pocket n is

$$S = 19\,000 - 100\,l/r \tag{13}$$

imum of 13 000 lb per sq in.

e's formula is sometimes referred to as *Gordon's formula, but Gordon used* teral dimension or the *diameter of the column instead of the least radius of*

The formulas used in the Catalogue of the Bethlehem Steel Comp

$$S = 16\,000 - 55\,l/r, \text{ for } l/r \text{ over } 55$$

and $$S = 13\,000 \text{ lb per sq in, for } l/r \text{ under } 55$$

Fowler's slightly modified formula for steel struts in trusses is

$$S = 12\,500 - 50\,l/r$$

The value 50 in Fowler's formula is $41\frac{2}{3}$ when l is in inches, an
is in feet.

For a comparison of most of these formulas, see Table XI, page
and the COMPARATIVE DIAGRAM OF FORMULAS, page 496.

14. Design of Steel Columns. Examples

Practical Use of Column-Formulas. Unlike the beam-formul
formulas in general use do not give a direct method of calculating t
of a column that will support a given load, owing to the presence i
formula of two unknown quantities, A and r, which are depend
another. Hence in designing columns, the section must be first
then tested for the safe load P, or for the maximum unit fiber-stre
an apparently roundabout method of designing columns, but
there seems to be no more direct way. When a column is to b
designed, its axial load P is given and also its length and the co
ends. A proper allowable unit stress, S, is assumed, suitable f
material and for the conditions under which it is to be used, or i
with the requirements of the local building code; or the value of
the specification according to which the column is to be design
section is then selected in accordance with the principles explained
to 469. For this assumed cross-section A and r are determined a
stituted in the formula, which is solved for P. If the assumed di
a value for P that agrees with the actual load, they are correct.
the resulting value of P is smaller than the actual load, the assum
small, and it will be necessary to choose a larger size and solve
the contrary, the actual load is less than the safe calculated load, a

ple Showing the Method of Selecting Bethlehem Rolled H Columns for Buildings

lustration, the interior columns of an actual sixteen-story building are taken as ple. The story-heights and the loads on the columns are given in the following on:

es	Heights of stories, ft	Loads on col-umns, tons	Safe loads, tons	H column-section required				
				Dimensions			Weights of sec-tions, lb per lin ft	Section-numbers
				D, in	T, in	B, in		
h	12	27	55.0	7⅞	7/16	8.00	31.5	H8
h h	13 14	53 79	81.5	8⅜	11/16	8.12	48.0	H8
h h	13 13	104 128	132.2	10⅜	13/16	10.12	71.0	H10
h h	13 13	151 174	174.8	12¼	⅞	12.08	91.5	H12
h h	13 13	197 219	219.1	14¼	15/16	14.08	114.5	H14
h h	13 13	241 261	263.8	14⅝	1⅛	14.19	138.0	H14
h h	13 13	281 301	310.1	15	1 5/16	14.31	162.0	H14
l l	13 15	321 341	341.3	15¼	1 7/16	14.39	178.5	H14
t nent	17 12	363 395	403.5	15¾	1 11/16	14.54	211.0	H14

the depth of the column, *T* the thickness of the flanges and *B* the breadth of the

mns for buildings are usually selected in lengths of two stories. By inspection of the of safe loads for H columns, it is found that no columns smaller than 14-in H sec- ave sufficient capacity for the lower stories. Where there is no limitation as to e of the column, the column with the largest dimensions and having the required y will be the most economical. The unsupported length of a column should not 150 radii of gyration, which is the limit of length for which safe loads are given tables. In the best practice the unsupported length of a column is frequently d not to exceed 120 or 125 times the least radius of gyration; various limits for indicated in the tables by zigzag lines. The safe loads given in the tables are for tric or symmetric loading. When the loads are not centrally or symmetrically l, the size of the column should be calculated by Formula (18), page 486.

mple 8. Suppose that in a 20-story office-building to be erected in Chicago, ad on each of the first-story columns, which are 16 feet in length, is 700 What columns should be used?

ning to Table XXI, page 515, giving the safe *loads for Bethlehem 14-in* umns it is seen *that a 14-in 287.5-lb column, the heaviest rolled, will t only 549.3 tons; this type of column, therefore, cannot be used.* More-

of 14 feet, the height of most of the upper stories, 469 000 lb; and a
umn (Table XXVI, page 541) composed of two 12-in 30-lb chann
14 by ¾-in plates will support 502 000 lb. The former weighs 125 a
131.4 lb per lin ft, so there is not much choice as far as economy o
concerned. The channel-column, however, requires four rows of
the plate-and-angle column requires only two rows, so this added
fabrication would have to be considered. Assuming, however, tha
and-angle type is more desirable, the next step is to design the
columns.

The load upon each of the uppermost columns, which are 20 ft i
70 000 lb. Turning to Table XXIV, page 518, it will be seen tha
composed of four 4 by 3 by ⅜-in angles and one 8 by ⅜-in web w
for a length of 20 ft, 77 000 lb; but this load is below the lower zig
hence the slenderness-ratio of the column exceeds 120. Assuming,
pose of illustration, that the limit of l/r is 120, a heavier sectio
selected. On page 519 of Table XXIV, continued, it is seen that
20-ft column, for which l/r does not exceed the required ratio, is on
of four 5 by 3½ by ⅜-in angles and one 10 by ⅜-in web, and that
of 20 ft it will support 121 000 lb, or 51 000 lb more than will come

Continuing the design of the columns, suppose that one in the
14 ft in length, supports 175 tons, or 350 000 lb. From Table XXI
it is found that a column composed of four 6 by 4 by ⅝-in angles an
½-in web-plate, for a length of 14 ft, will carry 373 000 lb. In th
safe load is calculated by Formula (13), whereas the Chicago Bu
specifies Formula (14). Hence, as this building is to be erected in
chosen column must be tested by the latter formula. Its A is 29.4
its least r, 2.65 in. To test it by the formula, $l = 14$ ft × 12 = 168 i
168 in/2.65 in = 63. Substituting in Formula (14), $S = 16\,000 -$ (
16 000 − 4 410 = 11 590 lb per sq in. From Formula (10), the safe
column, $P = AS$ = 29.44 sq in × 11 590 lb per sq in = 341 209 lb, w
than the actual load. Therefore the next heavier column, with a

$$= \frac{\quad}{16\,227} = \frac{\quad}{16\,227} = 11\,015 \text{ lb per sq in}$$

mula (10), page 481,

$= 14.37$ sq in × 11 015 lb per sq in = 158 285 lb or 79.1 tons,

than the tabular load. Hence the next heavier column, weighing
, would have to be used.

. Figure 7, page 342, shows the cross-section of one of the
mns in the Bankers' Trust Company's Building, New York City.
length and supports 2 230 tons. Is the column safe?

ep is to find its least radius of gyration which is equal to $\sqrt{I/A}$.
nent of inertia of this section was found to be 17 030. (See page
ea is made up as follows:

The flanges are composed of six 27 by $\frac{3}{4}$-in plates
y $\frac{11}{16}$-in plates. The area of the cross-section of
in plate is 20.25 sq in and of the six plates, 121.50 sq
of the section of each 27 by $\frac{11}{16}$-in plate is 18.56
e two plates, 37.12 sq in. Hence the total sectional
121.50 + 37.12 = 158.62 sq in

GLES. Each flange-angle is 6 by 6 by $\frac{15}{16}$ in. Its
s 10.38 sq in. Hence for the four, $A = 10.38 \times 4 =$ 41.52 sq in

B. The outer web-plates are each 18 by $\frac{11}{16}$ in.
ch one is 12.375 sq in and of the eight 99.00 sq in

h web-angle is 6 by $3\frac{1}{2}$ by $\frac{15}{16}$ in with a section-
q in; and for four angles the section-area is 32.12 sq in

web is composed of two 18 by $\frac{9}{16}$-in plates, each
-area of 10.125 sq in. For two the area is 20.25 sq in

f the entire section, therefore, is 351.51 sq in

$$^2 = I/A = 17\,030/351.5 = 48.5 \quad \text{and} \quad r = \sqrt{48.5} = 7 \text{ in}$$

$$l = 20 \text{ ft} = 240 \text{ in and } l/r = 240 \text{ in}/7 \text{ in} = 34.3$$

g in the New York City building code Formula (15), page 482, the bending moments caused by them in the column balance each o the CENTER OF STRESS may be considered as coinciding with the a column. When, however, a load is applied on one side (Fig. 25) corresponding load on the opposite side, it is called an ECCENTRIC LOA area of the cross-section of the column should be increased corresp There is unfortunately no direct method by which this additional ar determined. The usual method of procedure is to assume a section of that required to support the total load and then compute the fiber- to the combined balanced and eccentric loads. If this works out or too small another trial is made.

Formula for Eccentric Loads on Steel Columns. The followin (compare with Fig. 25) is used to the combined fiber-stresses due to th tric and eccentric loads:

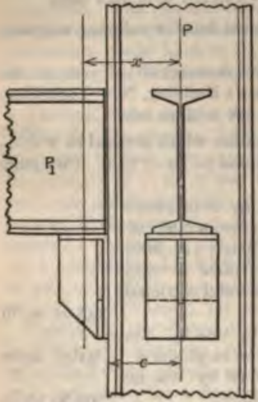


Fig. 25. Channel-column with Eccentric Load. Elevation

Let P = the concentric or balanced pounds,

P_1 = the eccentric load in pound

M = the bending moment du eccentric load in inch-poun

x = the eccentricity of the lo inches. (See note below.)

I = the moment of inertia of of the cross-section of th about an axis at right-ang direction of the bending,

c = the distance of the outermo the cross-section from the

A = the area of column-section inches and

S = the safe unit fiber-stress i per square inch.

Then

$$S = (P + P_1)/A + Mc/I$$

Note. In measuring the ECCENTRICITY, the distance, x, is generally from the axis of the column to the center line or half-breadth line of t or bearing.

Examples of Eccentric Loading of Steel Columns. The foll amples illustrate the use of the formula and tables in determining the safe eccentric loads for steel columns.

Example 11. The total load on the top of a column 32 ft in length is 194 000 lb, of which 30 000 lb come from the end of a girder. There is no corresponding load on the opposite side. (See Fig. 26.) It is proposed to use a channel-column. What is the size of the required column?

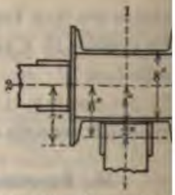


Fig. 26. Channel-col Eccentric Load. S

By referring to Table XXVI, page 539, it is seen that a column composed of two 12-in 20.5-lb channels and two 14 by ⅜-in plates will support, for a len ft, 227 000 lb, a somewhat greater load than will come on the colu the section of this column, $I_{1-1} = 415$, $A = 22.56$ sq in, $r = 4.29$ in

. Substituting in Formula (11), page 481, to find the safe unit

$$\frac{500}{\frac{}{00}(l/r)^2} = \frac{12\,500}{1+(89)^2/36\,000} = \frac{12\,500}{1+7\,921/36\,000} = \frac{12\,500}{43\,921/36\,000}$$

$$\frac{\times 36\,000}{921} = \frac{450\,000\,000}{43\,921} = 10\,245 \text{ lb per sq in}$$

stress in pounds per square inch of the column-section is found (18), $S = (P+P_1)/A + Mc/I$. $P = 164\,000$ lb, $P_1 = 30\,000$ lb, in and $M = Px$ in-lb. $x =$ the distance in inches from the axis lumn to the outside of the web, plus the distance from the outside the center of the bracket. The former distance can be found from . It is 4 in. Let the distance from the outside of the web of the e center of the bracket riveted to the web of the channel be 2 in, of the bracket being 4 in; then x, the lever-arm of the moment of or the eccentricity, is 4 in + 2 in = 6 in. M, therefore, is P_1x or in. c is 7 in, since the plates are 14 in wide. $I_{2-2} = 415$. Sub-ormula (18)

$$\frac{0+30\,000}{2.56} + \frac{30\,000 \times 6 \times 7}{415} = 8\,600 + 3\,036 = 11\,636 \text{ lb per sq in}$$

eeds the safe unit fiber-stress of 10 245 lb per sq in, the column- small.

d trial, consider a 12-in, 20.5-lb channel-column with 14 by ½-in this section, $I_{2-2} = 473$, $A = 26.06$ sq in, $r_{2-2} = 4.26$ in and $l/r =$.

$$= \frac{12\,500}{1+(90)^2/36\,000} = \frac{12\,500}{1+8\,100/36\,000} = \frac{12\,500}{44\,100/36\,000}$$

$$= \frac{12\,500 \times 36\,000}{44\,100} = 10\,204 \text{ lb per sq in.}$$

stress from Formula (18), as before, is

$$\frac{0+30\,000}{6.06} + \frac{30\,000 \times 6 \times 7}{473} = 7\,444 + 2\,664 = 10\,108 \text{ lb per sq in}$$

than the safe stress of 10 204 lb, the second selection is safe.

. A Bethlehem **H** column 14 ft long carries 90.56 tons, of which e eccentric, being applied to the flange of the column as shown e distance from the outside of the flange to the center of the 2 in. What is the size of the column required?

, 84.5-lb column, which, for a length of 14 ft, or 168 in, will carry able XX). For this column, $A = 24.92$, $r_{2-2} = 3.03$, $I_{1-1} = 676.1$, nd l is 14 ft, or 168 in; hence $l/r = 168/3.03 = 55$. Substituting 5), assuming that that formula is specified, $S = 15\,200 - 58 \times 55 =$ sq in. Since the eccentric load causes bending in a direction at o the axis 1–1, Fig. 27, the bending moment due to the eccentric 15.52 tons or 31 040 lb, multiplied by its lever arm x, which is the the axis 1–1 to the outside of the flange plus the distance from o the center of bearing. The former *dimension, taken from the* talogue, *is* $6\frac{1}{16}$ *in and the latter is 2 in; hence* $x = 8\frac{1}{16}$ in or for *in. The distance,* c*, also, of the outermost fiber from the axis*

1–1 is $6\frac{1}{16}$ in, which for convenience will b
axis 1–1 is 676.1. Substituting in Formula (

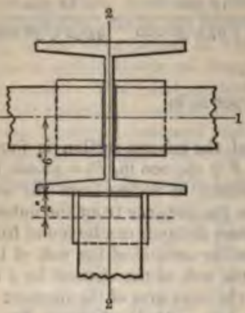


Fig. 27. Bethlehem H Column with Eccentric Load

(31 040 ×
lb per sq
stress of
too large,
64.5-lb col
Suppose,
load were
loads to
stress, as
no matter
unit stress
least radiu
exceeded.
loading th
bending a
Formula (
228.5, mu
2.25 in, o.
of the web. (See the Bethlehem Catalogue.
actual unit fiber-stress is $S = (150080 + 31040$
$= 7\,268 + 1\,835 = 9\,103$ lb per sq in.

16. Tables of Safe Loads

Safe Loads per Square Inch of Meta
Struts. To lessen the labor of calculating
struts, of whatever shape, the author has com
VALUES of S for ratios of l/r varying from 30
not whole numbers, the values can be readily
table should correspond exactly with the re
sponding formulas.

Safe Loads for Steel-Pipe Columns. T
LOADS for STEEL-PIPE COLUMNS. These loads
mended by the New York City Building
Steel-Pipe Columns, pages 469 to 474.)

Safe Loads for Channel and Angle-St
give the SAFE LOADS for standard CHANNELS a
those sizes that are most commonly used ar
LOADS for both the minimum and the maxim
If the strut is used also as a beam, or is stay
the larger value may be taken; but if free to
smaller value should be taken. If the strut
STRESS they should be computed as explaine
pages 571 and 572.

Safe Loads for Steel-Beam Columns, B
umns, Plate-and-Angle and Channel Col
giving the SAFE LOADS for these columns, w
but by the different manufacturers; they are,
safe, provided that an increase in area is made

Use of Table XI for Determining S
This table will be found of great assistance

b. In Example 10, the column in the Bankers' Trust Company
slenderness-ratio of 34.3, and an area of 351.5 sq in. The value
to 34, from column 5 of Table XI, is 13 228 and for 35 it is 13 170
hence for 34.3 it would be about 13 211 lb per in. Accordingly,
s 13 211 lb per sq in × 351.5 sq in = 4 643 666 lb. As an illustra-
lication to determine eccentric loads, refer again to Example 11.
r for this column is 89. The safe unit fiber-stress was found to be,
11), 10 245 lb per sq in. The practically identical result can be
oking for the value opposite 89 in column 2 of Table XI. It is
250 lb.

ble XI for Eccentric Loads for Steel Struts. To use Table
safe load for any steel strut take the following examples:

What is the safe resistance of a strut composed of two 5-in
separated ½ in and free to bend in either direction, the length of
7 ft 6 in?

rom Table XVIII, page 374, the least radius of gyration for this
ence $l/r = 90/1 = 90$. From the eighth column of Table XI, the
site 90 is 8 000 lb per sq in; the safe load, then, is equal to 8 000 lb
tiplied by the area of the two channels, 5.3 sq in, or 42 400 lb.

What is the safe stress for a 7-in 15-lb **I** beam when used as
90 in in length and free to bend in either direction.

rom Table IV, page 355, the least radius of gyration of this sec-
d the area is 4.42 sq in. $l/r = 90/0.78 = 115.4$. From the eighth
le XI, the value opposite 115 is 6 750 and opposite 116 it is 6 700
so for 115.4 it would be about 6 730 lb per sq in. The safe load,
730 lb per sq in × 4.42 sq in = 29 746 lb.

l the tables and rules given in Chapter X the SECTION-AREA and
OF GYRATION of any standard section or any combination of sec-
found; and once these are determined the strength of a strut or
e readily computed, as in the above examples.

of Floor-Loads Borne by Columns. (See, also, pages 148

tent and are made up of so many elements, that to avoid omissio
it is necessary to make a TABULATED LIST of all the loads transfe
the columns to the footings. In a building of skeleton construction
LOADS include floor and roof-loads, wind-loads, spandrel and pi
weight of the columns themselves and their fire-proof covering, and i
special loads, such as tanks, vaults, safes and elevator-loads. I
the FLOOR-LOADS it is advisable to separate the dead and live lo
venience in proportioning the footings. (See, also, pages 148 to 160.
for computing the WIND-LOADS on columns are given in Chapter X
loads, also, are considered as live loads. ECCENTRIC LOADS shou
tabulated separately from the balanced column-loads. On page
a form of COLUMN-SHEET which combines all ordinary requiren
TOTAL LOAD for each story is the sum of all of the loads above. Th
on page 492 shows a very convenient form for column-lengths and c

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* From Architectural Engineering, J. K. Freitag.

Form of Column-Sheet

y	Character of loading	Column No. 1		Column 2
		Load on column, concentric	Load on column, eccentric	
	Roof and ceiling, dead load......			
	Roof and ceiling, live load.......			
	Masonry piers....................			
	Spandrels, cornice, etc...........			
	Elevators.........................			
h	Tanks.............................			
g	Column and casing..............			
	Wind-load.........................			
	Total...........................			
	Sectional area required..........	sq in	sq in	
	From column above.*			
	Floor, dead load..................			
	Floor, live load...................			
	Masonry piers....................			
	Spandrels.........................			
	Safes, vaults, etc.................			
th	Column and casing..............			
	Wind-load.........................			
	Total...........................			
	Sectional area required..........	sq in	sq in	
	From column above.*			
	Floor, dead load..................			
	Floor, live load...................			
	Masonry piers....................			
	Spandrels.........................			
se-	Sidewalk..........................			
ent	Column and casing..............			
	Wind-load			
	Total...........................			
	Sectional area required..........	sq in	sq in	
	Deduct (½) live load...........			
tings	Total footing-load...........			
	Area of footing required.........	sq ft		

n bringing *down the load from the column above, the eccentric* loads may be added *concentric loads and their sum placed in the first column.*

Schedule of Column-Lengths and Parts

	Column No. 1		Column No. 2
Roof-line			
Top of columns	1′ 6½″		
7th story	½″	Four angles 4″×3″×5/16″ One plate 7″×5/16″	
7th Floor-line	23′ 4″		
6th Story			
6th Floor-line	2′ 2″	Four angles 5″×3″×3/8″ One plate 7″×3/8″	
5th Story	13′ 10″		
	¾″		
5th Floor-line	4¼″		
	¾″		
1st Floor-line	1′ 2¼″		
Basement	11′ 2″	Four Z's 4″×3/8″ One plate 7″×3/8″	
Top of stool			
Grade 15.0	8¼″		

XI. Safe Loads in Pounds per Square Inch of Metal-Area for Steel Columns and Struts

l = length in inches r = least radius of gyration in inches

Rankine's (Gordon's) and Cambria	Philadelphia	Boston	New York	Chicago	Am. Bridge Co. and Carnegie	Fowler's for struts	l/r
$\frac{12\,500}{1+\frac{l^2}{36\,000\,r^2}}$	$\frac{16\,250}{1+\frac{l^2}{11\,000\,r^2}}$	$\frac{16\,000}{1+\frac{l^2}{20\,000\,r^2}}$	$15\,200 - 58\,l/r$	$16\,000 - 70\,l/r$ 14 000 max	$19\,000 - 100\,l/r$ 13 000 max	$12\,500 - 50\,l/r$	
II	III	IV	V	VI	VII	VIII	IX
12 195	15 020	15 310	13 460	13 900	13 000	11 000	30
12 170	14 945	15 265	13 402	13 830	13 000	10 950	31
12 155	14 865	15 220	13 344	13 760	13 000	10 900	32
12 135	14 785	15 175	13 286	13 690	13 000	10 850	33
12 110	14 705	15 125	13 228	13 620	13 000	10 800	34
12 090	14 620	15 075	13 170	13 550	13 000	10 750	35
12 065	14 535	15 025	13 112	13 480	13 000	10 700	36
12 045	14 450	14 975	13 054	13 410	13 000	10 650	37
12 020	14 365	14 925	12 996	13 340	13 000	10 600	38
11 995	14 275	14 870	12 938	13 270	13 000	10 550	39
11 970	14 185	14 815	12 880	13 200	13 000	10 500	40
11 945	14 095	14 760	12 822	13 130	13 000	10 450	41
11 920	14 005	14 705	12 764	13 060	13 000	10 400	42
11 890	13 915	14 650	12 706	12 990	13 000	10 350	43
11 860	13 820	14 590	12 648	12 920	13 000	10 300	44
11 835	13 725	14 530	12 590	12 850	13 000	10 250	45
11 805	13 630	14 470	12 532	12 780	13 000	10 200	46
11 780	13 535	14 410	12 474	12 710	13 000	10 150	47
11 750	13 440	14 350	12 416	12 640	13 000	10 100	48
11 720	13 340	14 285	12 358	12 570	13 000	10 050	49
11 690	13 240	14 220	12 300	12 500	13 000	10 000	50
11 660	13 145	14 160	12 242	12 430	13 000	9 950	51
11 620	13 045	14 095	12 184	12 360	13 000	9 900	52
11 595	12 945	14 030	12 126	12 290	13 000	9 850	53
11 565	12 845	13 965	12 068	12 220	13 000	9 800	54
11 530	12 745	13 900	12 010	12 150	13 000	9 750	55
11 500	12 645	13 835	11 952	12 080	13 000	9 700	56
11 465	12 545	13 770	11 894	12 010	13 000	9 650	57
11 430	12 445	13 700	11 836	11 940	13 000	9 600	58
11 400	12 345	13 630	11 778	11 870	13 000	9 550	59
11 365	12 240	13 560	11 720	11 800	13 000	9 500	60
11 330	12 140	13 490	11 662	11 730	12 900	9 450	61
11 295	12 040	13 420	11 604	11 660	12 800	9 400	62
11 260	11 940	13 350	11 546	11 590	12 700	9 350	63
11 225	11 840	13 280	11 488	11 520	12 600	9 300	64
11 185	11 740	13 210	11 430	11 450	12 500	9 250	65
11 150	11 640	13 140	11 372	11 380	12 400	9 200	66
11 115	11 540	13 070	11 314	11 310	12 300	9 150	67
11 080	11 440	13 000	11 256	11 240	12 200	9 100	68
11 040	11 340	12 925	11 198	11 170	12 100	9 050	69

Table XI (Continued). Safe Loads in Pounds per Square Inch
Area for Steel Columns and Struts

l = length in inches, r = least radius of gyration in inche

l/r	Rankine's (Gordon's) and Cambria	Philadelphia	Boston	New York	Chicago	Am. Bridge Co. and Carnegie
	$\frac{12\,500}{1+\frac{l^2}{36\,000\,r^2}}$	$\frac{16\,250}{1+\frac{l^2}{11\,000\,r^2}}$	$\frac{16\,000}{1+\frac{l^2}{20\,000\,r^2}}$	$15\,200 - 58\,l/r$	$16\,000 - 70\,l/r$ 14 000 max	$19\,000 - 100\,l/r$ 13 000 max
I	II	III	IV	V	VI	VII
70	11 000	11 240	12 850	11 140	11 100	12 000
71	10 965	11 140	12 780	11 082	11 030	11 900
72	10 930	11 040	12 710	11 024	10 960	11 800
73	10 890	10 940	12 640	10 966	10 890	11 700
74	10 850	10 845	12 565	10 908	10 820	11 600
75	10 810	10 750	12 490	10 850	10 750	11 500
76	10 770	10 655	12 420	10 792	10 680	11 400
77	10 735	10 560	12 345	10 734	10 610	11 300
78	10 695	10 465	12 270	10 676	10 540	11 200
79	10 655	10 370	12 195	10 618	10 470	11 100
80	10 615	10 275	12 120	10 560	10 400	11 000
81	10 575	10 180	12 045	10 502	10 330	10 900
82	10 535	10 085	11 970	10 444	10 260	10 800
83	10 495	9 990	11 895	10 386	10 190	10 700
84	10 450	9 800	11 825	10 328	10 120	10 600
85	10 410	9 810	11 755	10 270	10 050	10 500
86	10 370	9 720	11 680	10 212	9 980	10 400
87	10 330	9 630	11 605	10 154	9 910	10 300
88	10 290	9 540	11 530	10 096	9 840	10 200
89	10 250	9 450	11 460	10 038	9 770	10 100
90	10 205	9 360	11 390	9 980	9 700	10 000
91	10 165	9 370	11 315	9 922	9 630	9 900
92	10 125	9 285	11 240	9 864	9 560	9 800
93	10 085	9 200	11 165	9 806	9 490	9 700
94	10 040	9 115	11 095	9 748	9 420	9 600
95	9 995	8 930	11 025	9 690	9 350	9 500
96	9 955	8 845	10 950	9 632	9 280	9 400
97	9 915	8 760	10 880	9 574	9 210	9 300
98	9 875	8 675	10 810	9 516	9 140	9 200
99	9 830	8 590	10 740	9 458	9 070	9 100
100	9 785	8 510	10 670	9 400	9 000	9 000
101	9 740	8 430	10 595	9 342	8 930	8 900
102	9 695	8 350	10 525	9 284	8 860	8 800
103	9 650	8 270	10 455	9 226	8 790	8 700
104	9 610	8 190	10 385	9 168	8 720	8 600
105	9 570	8 115	10 315	9 110	8 650	8 500
106	9 525	8 040	10 245	9 052	8 580	8 400
107	9 480	7 965	10 175	8 994	8 510	8 300
108	9 435	7 890	10 105	8 936	8 440	8 200
109	9 395	7 815	10 035	8 878	8 370	8 100

ontinued). Safe Loads in Pounds per Square Inch of Metal-Area for Steel Columns and Struts

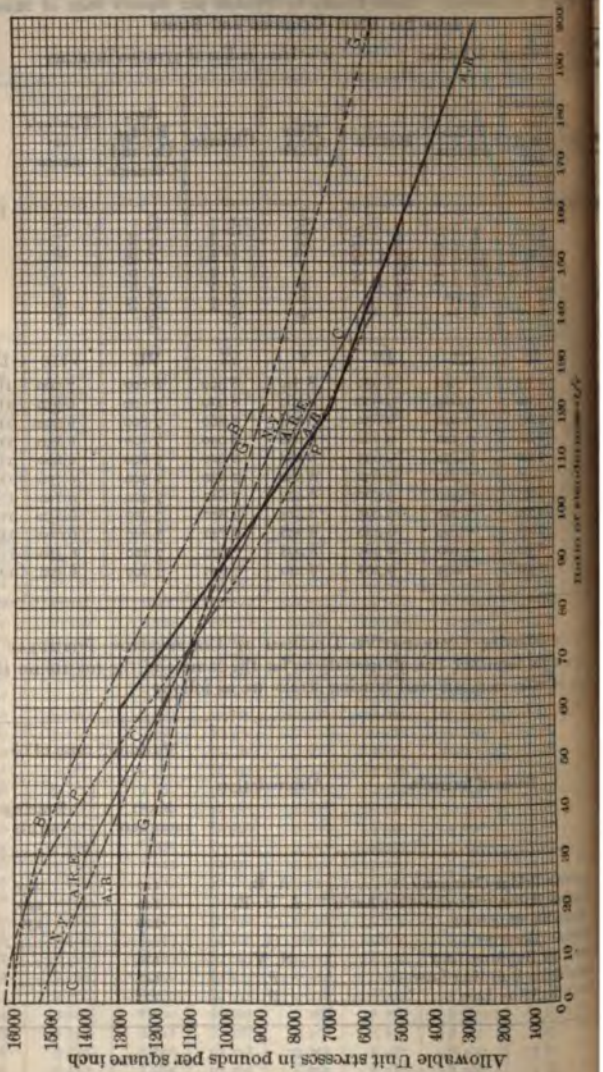
ngth in inches r = least radius of gyration in inches

's ia	Phila-delphia	Boston	New York	Chicago	Am. Bridge Co. and Carnegie	Fowler's for struts	l/r
30 000 r	$\dfrac{16\,250}{1+\dfrac{l^2}{11\,000\,r^2}}$	$\dfrac{16\,000}{1+\dfrac{l^2}{20\,000\,r^2}}$	$15\,200 - 58\,l/r$	$16\,000 - 70\,l/r$ $14\,000$ max	$19\,000 - 100\,l/r$ $13\,000$ max	$12\,500 - 50\,l/r$	
	III	IV	V	VI	VII	VIII	IX
	7 740	9 970	8 820	8 300	8 000	7 000	110
	7 665	9 900	8 762	8 230	7 900	6 950	111
	7 590	9 830	8 704	8 160	7 800	6 900	112
	7 520	9 760	8 646	8 090	7 700	6 850	113
	7 450	9 695	8 588	8 020	7 600	6 800	114
	7 380	9 630	8 530	7 950	7 500	6 750	115
	7 310	9 560	8 472	7 880	7 400	6 700	116
	7 240	9 495	8 414	7 810	7 300	6 650	117
	7 170	9 430	8 356	7 740	7 200	6 600	118
	7 100	9 365	8 298	7 670	7 100	6 550	119
	7 035	9 300	8 240	7 600	7 000	6 500	120

wing COMPARATIVE DIAGRAM OF COMPRESSION FORMULAS the ormulas, the abbreviations for the same and the maximum ratio members and bracing struts are as follows:

me of formula	Abbreviation	Maximum ratio of l/r	
		Main members	Bracing struts
dge Company	A. B.	120	200
lway Engineering Ass'n	A. R. E.	100	120
ling Law	C.	120	150
rdon)	G.	....	....
ilding Law	N. Y.	120	....
Building Law	P.	140	....
ng Law	B.	120	....

Comparative Diagram of Compression-Formulas



e XII.* Safe Loads in Tons of 2 000 Pounds for Standard Steel-Pipe Columns

ds in tons of 2 000 pounds. Table based on New York City building laws. For-
ised, $S = 15\,200 - 58\,l/r$, in which

S = allowable compressive stress for steel in pounds per square inch,
l = length of column in inches,
r = least radius of gyration in inches.

s above or to the left of the zigzag lines correspond to values of l/r greater than 120.

	Sizes of pipe. Diameters in inches								
hs	2	2½	3	3½	4	4½	5	6	7
	Thickness in decimal parts of an inch								
	0.154	0.203	0.216	0.226	0.237	0.247	0.258	0.280	0.301
	....	....	......	......	......	......	......	......	......
	....	....	......	......	......	......	......	......	19.16
	....	....	......	......	......	......	......	13.87	21.95
	....	....	......	......	......	......	......	16.47	24.74
	....	....	......	......	......	......	11.16	19.06	27.53
	....	....	......	......	......	9.72	13.55	21.66	30.32
	....	....	......	......	8.02	11.25	15.15	23.39	32.18
	....	....	......	6.41	9.49	12.78	16.74	25.12	34.04
	....	....	......	7.81	10.95	14.30	18.34	26.85	35.90
	....	....	6.27	9.20	12.42	15.83	19.93	28.58	37.76
	....	4.19	7.61	10.60	13.88	17.35	21.52	30.31	39.62
	....	4.81	8.27	11.30	14.61	18.11	22.32	31.17	40.55
	....	5.44	8.94	11.99	15.34	18.88	23.12	32.04	41.48
	2.94	6.07	9.61	12.69	16.07	19.64	23.91	32.90	42.41
	3.42	6.69	10.27	13.39	16.81	20.40	24.71	33.77	43.34
	3.89	7.32	10.94	14.09	17.54	21.17	25.51	34.63	44.27
	4.37	7.94	11.60	14.78	18.27	21.93	26.30	35.50	45.20
	4.84	8.57	12.27	15.48	19.00	22.69	27.10	36.36	46.13
	5.32	9.20	12.94	16.18	19.73	23.45	27.90	37.23	47.06
	5.79	9.82	13.60	16.88	20.46	24.22	28.69	38.09	47.99

	Sizes of pipe. Diameters in inches							
hs	8	9	10	11	12	13	14	15
	Thickness in decimals parts of an inch							
	0.322	0.342	0.365	0.375	0.375	0.375	0.375	0.375
	24.04	33.53	45.38	55.49	64.44	75.63	84.58	93.53
	28.02	37.76	49.90	60.12	69.07	80.26	89.21	98.17
	31.00	40.93	53.28	63.60	72.55	83.74	92.69	101.64
	33.99	44.10	56.66	67.08	76.03	87.22	96.17	105.12
	36.97	47.27	60.05	70.55	79.51	90.69	99.65	108.60
	39.96	50.44	63.43	74.03	82.98	94.17	103.12	112.08
	41.95	52.55	65.69	76.35	85.30	96.49	105.44	114.40
	43.94	54.66	67.94	78.67	87.62	98.81	107.76	116.71
	45.93	56.78	70.20	80.99	89.94	101.13	110.08	119.03
	47.92	58.89	72.46	83.30	92.26	103.45	112.40	121.35
	49.90	61.01	74.71	85.62	94.57	105.76	114.72	123.67
3	50.90	62.06	75.84	86.78	95.73	106.92	115.88	124.83
2	51.89	63.12	76.97	87.94	96.89	108.08	117.03	125.99
1	52.89	64.18	78.10	89.10	98.05	109.24	118.19	127.15
0	53.88	65.23	79.22	90.26	99.21	110.40	119.35	128.31
9	54.88	66.29	80.35	91.42	100.37	111.56	120.51	129.47
8	55.87	67.35	81.48	92.57	101.53	112.72	121.67	130.62
7	56.87	68.40	82.61	93.73	102.69	113.88	122.83	131.78
6	57.86	69.46	83.74	94.89	103.85	115.04	*123.99*	*132.94*
5	58.86	70.52	84.86	*96.05*	*105.00*	*116.20*	*125.15*	*134.10*

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* *Furnished by the National Tube Company, Pittsburgh, Pa.*

Table XIII.* **Safe Loads in Tons of 2 000 Pounds for Extra-Stro
Pipe Columns**

Loads in tons of 2 000 pounds. Table based on New York City building mula used, $S = 15\,200 - 58\,l/r$, in which

S = allowable compressive stress for steel in pounds per square inch,
l = length of column in inches,
r = least radus of gyration in inches.

Loads above or to the left of the zigzag lines correspond to values of l/r grea

Lengths, ft	Sizes of pipe. Diameters in inches							
	2	2½	3	3½	4	4½	5	6
	Thickness in decimal parts of an inch							
	0.218	0.276	0.300	0.318	0.337	0.355	0.375	0.
40	......	......	......	......	......	......	......	...
36	......	......	......	......	......	......	......	...
33	......	......	......	......	......	......	......	19.
30	......	......	......	......	......	......	......	23.
27	......	......	......	......	......	......	15.22	27.
24	......	......	......	......	......	13.10	18.69	31.
22	......	......	......	......	10.65	15.29	21.01	34.
20	......	......	......	8.36	12.72	17.48	23.32	37.
18	......	......	......	10.32	14.80	19.67	25.63	39.
16	......	......	8.14	12.28	16.88	21.86	27.95	42.
14	......	5.25	9.99	14.24	18.95	24.05	30.26	45.
13	......	6.09	10.91	15.22	19.99	25.14	31.42	46
12	......	6.94	11.84	16.20	21.03	26.24	32.57	47
11	3.85	7.79	12.76	17.18	22.07	27.33	33.73	49
10	4.52	8.64	13.68	18.16	23.11	28.43	34.89	50
9	5.19	9.49	14.61	19.14	24.15	29.52	36.04	51
8	5.86	10.34	15.53	20.12	25.19	30.61	37.20	53
7	6.53	11.19	16.46	21.10	26.23	31.71	38.35	54
6	7.20	12.03	17.38	22.08	27.26	32.80	39.51	55
5	7.87	12.88	18.30	23.06	28.30	33.90	40.67	57

Lengths, ft	Sizes of pipe. Diameters in inches						
	8	9	10	11	12	13	14
	Thickness in decimal parts of an incn						
	0.500	0.500	0.500	0.500	0.500	0.500	0.50
40	35.27	47.18	60.59	72.52	84.45	99.36	111.
36	41.44	53.36	66.77	78.70	90.63	105.54	117.
33	46.07	57.99	71.40	83.33	95.26	110.18	122.
30	50.70	62.62	76.04	87.97	99.90	114.81	126.
27	55.33	67.25	80.67	92.60	104.53	119.45	131.
24	59.96	71.88	85.30	97.23	109.17	124.08	136.
22	63.05	74.97	88.39	100.32	112.25	127.17	139.
20	66.13	78.06	91.48	103.41	115.34	130.26	142.
18	69.22	81.15	94.57	106.50	118.43	133.35	145.
16	72.31	84.23	97.66	109.59	121.52	136.44	148.
14	75.39	87.32	100.74	112.68	124.61	139.53	151.
13	76.94	88.87	102.29	114.22	126.16	141.08	153.
12	78.48	90.41	103.83	115.77	127.70	142.62	154.
11	80.02	91.95	105.38	117.31	129.25	144.17	156.
10	81.56	93.50	106.92	118.86	130.79	145.71	157.
9	83.11	95.04	108.47	120.40	132.34	147.26	159.
8	84.65	96.58	110.01	121.95	133.88	148.80	160.
7	86.19	98.13	111.56	123.49	135.43	150.35	162.
6	87.74	99.67	113.10	125.04	136.97	151.89	163.
5	89.28	101.22	114.64	126.58	138.52	153.44	165.

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* Furnished by the National Tube Company, Pittsburgh, P

e XIV. Safe Loads in Tons of 2 000 Pounds for Struts Formed of a Pair of Steel Channels

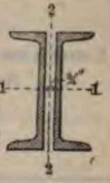
Distance between webs, ¾ in

rut is free to bend in either direction, use smaller load given

Stresses in pounds per square inch:

12 000 for lengths of 30 radii and under;

13 500 − 50 l/r for lengths over 30 radii



th. n	Weight per lin foot, lb*	Thickness of web, in	Area of two channels, sq in	$r_{2\text{-}2}$ $r_{1\text{-}1}$, in	Length in feet					
					8	9	10	11	12	14
	33	0.40	19.80	1.48	101.57	97.56	93.55	89.54	85.48	77.44
				5.62	118.80	118.80	118.80	118.80	118.80	118.80
	35	0.43	20.58	1.47	105.32	101.13	96.93	92.73	88.54	80.09
				5.58	123.48	123.48	123.48	123.48	123.48	123.00
	40	0.52	23.52	1.46	120.13	115.30	110.48	103.66	100.78	91.14
5				5.43	141.12	141.12	141.12	141.12	141.12	140.41
	45	0.62	26.48	1.45	134.91	129.48	123.99	118.50	113.00	102.08
				5.32	158.88	158.88	158.88	158.88	158.88	157.82
	50	0.72	29.42	1.46	150.36	144.23	138.20	132.17	126.06	114.00
				5.23	176.52	176.52	176.52	176.52	176.52	174.75
	55	0.82	32.36	1.47	165.60	159.00	152.40	145.78	139.22	126.10
				5.16	194.16	194.16	194.16	194.16	194.16	192.00
	20½	0.28	12.06	1.34	59.81	57.10	54.40	51.70	49.02	43.62
				4.61	72.36	72.36	72.36	72.36	71.99	70.43
	25	0.39	14.70	1.31	72.32	68.95	65.60	62.21	58.83	52.03
				4.43	88.20	88.20	88.20	88.20	87.28	85.26
	30	0.51	17.64	1.30	86.52	82.46	78.36	74.30	70.25	62.09
2				4.28	105.84	105.84	105.84	105.48	104.25	101.78
	35	0.64	20.58	1.31	101.25	96.52	91.78	87.10	82.37	72.90
				4.17	123.48	123.48	123.48	122.65	121.16	118.33
	40	0.76	23.52	1.32	116.01	110.66	105.31	99.96	94.66	83.96
				4.09	141.12	141.12	141.12	139.82	138.06	134.65
	15	0.24	8.92	1.24	42.94	40.78	38.64	36.48	34.32	30.01
				3.87	53.52	53.52	53.29	52.62	51.91	50.43
	20	0.38	11.76	1.20	55.86	52.92	49.98	47.04	44.10	38.22
				3.66	70.56	70.56	69.73	68.79	67.82	65.85
	25	0.53	14.70	1.20	69.82	66.15	62.47	58.80	55.12	47.77
				3.52	88.20	87.94	86.69	85.44	84.19	81.69
	30	0.68	17.64	1.22	84.40	80.04	75.71	71.35	67.03	58.34
				3.42	105.84	105.13	103.63	102.04	100.20	97.41
	35	0.82	20.58	1.26	99.76	94.82	89.93	85.04	80.16	70.33
				3.35	123.48	122.34	120.49	118.64	116.79	113.13
	13¼	0.23	7.78	1.19	36.83	34.87	32.91	30.94	28.98	25.07
				3.49	46.68	46.48	45.82	45.16	44.50	43.15
	15	0.29	8.82	1.17	41.45	39.18	36.93	34.66	32.41	27.89
				3.40	52.92	52.52	51.81	50.98	50.10	48.64
	20	0.45	11.76	1.15	54.85	51.77	48.71	45.65	42.57	36.42
				3.21	70.56	69.50	68.38	67.29	66.23	64.00
	25	0.62	14.70	1.17	69.09	65.31	61.55	57.77	54.00	46.48
				3.10	87.83	86.43	85.00	83.56	82.17	79.30

* *Of single channel.*

Table XIV (Continued). Safe Loads in Tons of 2 000 Pounds for Formed of a Pair of Steel Channels

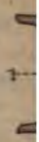
Distance between webs, ½ in

If strut is free to bend in either direction, use smaller load given

Stresses in pounds per square inch:

11 000 for lengths of 50 radii and under;

13 500 − 50 l/r for lengths over 50 radii



Depth, in	Weight per lin foot, lb*	Thickness of web, in	Area of two channels, sq in	r_{2-2} r_{1-1}, in	Length in feet			
					6	7	8	9
8	11.25	0.22	6.70	1.04	33.63	31.70	29.76	27.83
				3.11	36.85	36.85	36.85	36.85
	13.75	0.31	8.08	1.04	40.56	38.23	35.89	33.57
				2.98	44.44	44.44	44.44	44.44
	16.25	0.40	9.56	1.03	47.82	45.05	42.25	39.48
				2.89	52.58	52.58	52.58	52.58
	18.75	0.49	11.02	1.03	55.12	51.93	48.70	45.51
				2.82	60.61	60.61	60.61	60.61
	21.25	0.58	12.50	1.03	62.53	58.90	55.25	51.62
				2.77	68.75	68.75	68.75	68.75
7	9.75	0.21	5.70	0.99	28.11	26.39	24.66	22.94
				2.72	31.35	31.35	31.35	31.35
	12.25	0.32	7.20	0.99	35.51	33.33	31.15	28.98
				2.59	39.60	39.60	39.60	39.60
	14.75	0.42	8.68	0.99	42.71	40.18	37.56	34.93
				2.50	47.74	47.74	47.74	47.74
	17.25	0.53	10.14	1.00	50.19	47.15	44.10	41.06
				2.44	55.77	55.77	55.77	55.77
	19.75	0.63	11.62	1.00	57.52	54.03	50.54	47.06
				2.39	63.91	63.91	63.91	63.91
6	8.00	0.20	4.76	0.94	23.02	21.50	19.98	18.46
				2.34	26.18	26.18	26.18	26.18
	10.50	0.32	6.18	0.94	29.89	27.91	25.94	23.97
				2.21	33.99	33.99	33.99	33.99
	13.00	0.44	7.64	0.95	37.11	34.68	32.27	29.87
				2.13	42.02	42.02	42.02	41.88
	15.50	0.56	9.12	0.95	44.30	41.40	38.53	35.66
				2.07	50.16	50.16	50.16	49.68
5	6.50	0.19	3.90	0.89	18.43	17.13	15.81	14.49
				1.95	21.45	21.45	21.45	20.92
	9.00	0.33	5.30	0.90	25.17	23.41	21.65	19.87
				1.83	29.15	29.15	28.83	27.97
	11.50	0.48	6.76	0.91	32.26	30.03	27.81	25.58
				1.75	37.18	37.18	36.36	35.20
4	5.25	0.18	3.10	0.84	14.28	13.17	12.07	10.96
				1.56	17.05	16.75	16.15	15.55
	6.25	0.25	3.68	0.84	16.95	15.64	14.33	13.02
				1.51	20.24	19.72	18.99	18.26
	7.25	0.32	4.26	0.84	19.62	18.10	16.59	15.07
				1.46	23.43	22.63	21.75	20.87

* Of single channel.

ble XV. Safe Loads in Tons of 2 000 Pounds for Single-Steel-Angle Struts

ANGLES WITH UNEQUAL LEGS

Stresses in pounds per square inch:
11 000 for lengths of 50 radii and under;
13 500 − 50 l/r for lengths over 50 radii

e,	Thick-ness, in	r axis 3-3,* in	Area, sq in	Length in feet						
				4	5	6	7	8	9	10
4	⅜	0.88	3.61	....	....	....	....	....	....	....
	⅞	0.86	7.99	42.78	40.00	37.21	34.44	31.64	28.86	26.07
3½	⅜	0.76	3.05	....	....	....	....	....	....	....
	¾	0.75	5.81	....	....	....	....	....	....	....
3	5/16	0.66	2.40	....	....	....	....	....	....	....
	¾	0.64	5.44	....	....	....	....	....	....	....
3	5/16	0.66	2.25	....	....	....	....	....	....	....
	¾	0.64	5.06	24.66	22.29	19.92	17.55	15.18	....	....
3½	5/16	0.73	2.25	11.49	10.57	9.65	8.72	7.79	6.86	....
	¾	0.72	5.06	25.73	23.62	21.51	19.40	17.29	15.18	....
3	5/16	0.65	2.09	10.25	9.28	8.32	7.36	6.39	....	....
	¾	0.64	4.69	22.86	20.67	18.47	16.27	14.07	....	....
	5/16	0.63	1.93	9.35	8.43	7.51	6.59	....	....	....
3	⅜	0.62	2.30	11.07	9.96	8.84	7.74	....	....	....
	⅝	0.62	3.67	17.67	15.90	14.12	12.35	....	....	....
	¼	0.54	1.44	6.52	5.72	4.92	....	....	....	....
2½	⅜	0.54	2.11	9.55	8.38	7.21	....	....	....	....
	½	0.53	2.75	12.34	10.78	9.22	....	....	....	....
	¼	0.53	1.31	5.88	5.13	4.39	....	....	....	....
½	⅜	0.52	1.92	8.52	7.42	6.31	....	....	....	....
	½	0.52	2.50	11.10	9.66	8.22	....	....	....	....
	¼	0.43	1.19	4.71	3.88	....	....	....	....	....
	⅜	0.43	1.73	6.85	5.64	....	....	....	....	....
	½	0.43	2.25	8.91	7.34	....	....	....	....	....
	¼	0.42	1.06	4.13	3.37	....	....	....	....	....
	⅜	0.42	1.55	6.03	4.93	....	....	....	....	....
	½	0.42	2.00	7.79	6.36	....	....	....	....	....

is the least radius of gyration with reference to the diagonal axis 3-3. (See
l, pages 362 to 365.)

Table XV (Continued). Safe Loads in Tons of 2 000 Pounds for Single Steel-Angle Struts

ANGLES WITH EQUAL LEGS

Stresses in pounds per square inch:
11 000 for lengths of 50 radii and under;
13 500 − 50 l/r for lengths over 50 radii

Size, in	Thickness, in	r axis 3-3,* in	Area, sq in	Length in feet						
				4	5	6	7	8	9	[illegible]
6 ×6	3/8	1.19	4.36	23.98	23.93	22.83	21.74	20.64	19.54	[illegible]
	5/8	1.18	7.11	39.10	38.96	37.14	35.35	33.54	31.72	[illegible]
	7/8	1.17	9.74	53.57	53.27	50.77	48.28	45.77	43.26	[illegible]
5 ×5	3/8	0.99	3.61	19.85	18.89	17.80	16.71	15.64	14.53	[illegible]
	5/8	0.97	5.86	32.23	30.50	28.68	26.86	25.06	23.24	[illegible]
	7/8	0.96	7.99	43.94	41.44	38.95	36.45	33.95	31.45	[illegible]
4 ×4	3/8	0.79	2.86	14.96	13.88	12.79	11.71	10.61	9.53	[illegible]
	1/2	0.78	3.75	19.54	18.10	16.65	15.22	13.78	12.33	[illegible]
	5/8	0.77	4.61	23.93	22.13	20.33	18.55	16.75	14.95	[illegible]
	3/4	0.77	5.44	28.24	26.12	23.99	21.89	19.77	17.65	[illegible]
3½×3½	5/16	0.69	2.09	10.47	9.56	8.65	7.74	6.83	....	[illegible]
	1/2	0.68	3.25	16.20	14.77	13.34	11.90	10.47	....	[illegible]
	5/8	0.67	3.98	19.74	17.95	16.17	14.39	12.61	....	[illegible]
	3/4	0.67	4.69	13.26	21.16	19.06	16.96	14.86	....	[illegible]
3 ×3	1/4	0.59	1.44	6.79	6.06	5.32	4.59	....	....	[illegible]
	3/8	0.58	2.11	9.88	8.78	7.69	6.60	....	....	....
	1/2	0.58	2.75	12.87	11.45	10.03	8.60	....	....	[illegible]
	5/8	0.57	3.36	15.60	13.84	12.07	10.30	....	....	[illegible]
2½×2½	3/16	0.49	0.90	3.87	3.32	2.76	....	....	....	[illegible]
	1/4	0.49	1.19	5.10	4.39	3.66	....	....	....	[illegible]
	3/8	0.48	1.73	7.35	6.27	5.19	....	....	....	....
	1/2	0.47	2.25	9.44	8.01	6.57	....	....	....	....
2¼×2¼	3/16	0.44	0.81	3.26	2.70	....	....	....	....	....
	1/4	0.44	1.06	4.26	3.54	2.80	....	....	....	....
	3/8	0.43	1.55	6.13	5.05	3.95	....	....	....	....
	7/16	0.43	1.78	7.14	5.80	4.53	....	....	....	....
2 ×2	3/16	0.40	0.72	2.70	2.16	....	....	....	....	....
	1/4	0.39	0.94	3.45	2.72	2.00	....	....	....	....

* This is the least radius of gyration, with reference to the diagonal axis 3-3. (See Table XII, pages 366 and 367.)

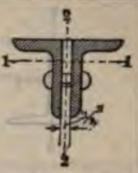
XVI. Safe Loads in Tons of 2 000 Pounds for Double-Steel-Angle Struts

LEGS PARALLEL AND ONE-HALF INCH APART

Stresses in pounds per square inch:

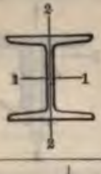
11 000 for lengths of 50 radii and under;

13 500 − 50 l/r for lengths over 50 radii



Thickness, in	Least r, in	Area two angles, sq in	Length in feet 5	6	7	8	10	11	12
1/2	2.49	13.52	74.36	74.36	74.36	74.36	74.36	73.34	71.72
1	2.65	26.82	147.51	147.51	147.51	147.51	147.51	147.51	144.62
3/8	1.67	7.22	39.71	39.71	39.65	38.35	35.77	34.47	33.14
13/16	1.74	14.94	82.17	82.17	82.17	80.26	75.07	72.50	69.91
3/8	1.43	6.84	37.62	37.57	36.13	34.69	31.82	30.38	29.00
1/2	1.46	9.00	49.50	49.50	47.81	45.97	42.27	40.41	38.56
5/8	1.49	11.10	61.05	61.05	59.27	57.05	52.51	50.29	48.07
13/16	1.52	14.12	77.66	77.66	75.82	73.03	67.42	64.65	61.88
3/8	1.59	6.46	35.53	35.53	35.07	33.86	31.41	30.20	28.99
3/4	1.54	12.38	68.09	68.09	66.69	64.28	59.45	57.04	54.62
3/8	1.51	6.10	33.55	33.55	32.70	31.49	29.05	27.84	26.64
3/4	1.55	11.62	63.91	63.91	62.69	60.45	55.95	53.71	51.44
3/8	1.27	5.72	31.46	30.50	29.15	27.80	25.09	23.75	22.39
1/2	1.30	7.50	41.25	40.23	38.51	36.78	33.32	31.59	29.85
5/8	1.33	9.22	50.71	49.76	47.69	45.59	41.40	39.30	37.29
3/4	1.36	10.88	59.84	58.94	56.63	54.23	49.42	47.05	44.66
3/8	1.25	5.34	29.37	28.35	27.07	25.79	23.23	21.94	20.66
3/4	1.20	10.12	55.66	53.13	50.60	48.07	43.01	40.48	39.95
3/8	1.26	4.96	27.28	26.28	25.12	24.00	21.64	20.49	19.30
3/4	1.22	9.38	51.59	49.47	47.18	44.85	40.24	37.94	35.64
1/4	1.12	2.88	15.58	14.81	14.03	13.26	11.72	10.95	10.18
3/8	1.10	4.22	22.73	21.59	20.42	19.28	16.97	15.82	14.67
1/2	1.09	5.50	29.56	28.05	26.53	25.02	21.98	20.47	18.96
11/16	1.06	7.30	38.92	36.86	34.78	32.74	28.58	26.51	24.43
1/4	0.93	2.38	12.22	11.45	10.69	9.92	8.39	7.62	....
1/2	0.92	4.50	23.04	21.58	20.10	18.64	15.70	14.23	....
3/16	0.79	1.62	7.86	7.24	6.63	6.01	....	....	....
1/2	0.75	4.00	19.00	17.40	15.80	14.20	....	....	....
3/16	0.62	1.44	6.22	5.54	4.82	4.13	....	....	....
1/4	0.61	1.88	8.08	7.14	6.20	5.26	....	....	....

Table XVII.* **Safe Loads in Units of 1 000 Pounds for Steel-Be Columns**



Allowable fiber-stress in pounds per square inc for lengths of 60 radii or under

Reduced for lengths over 60 radii by Formula (13)

$$S = 19\,000 - 100\, l/r$$

Weights do not include details

Effective length, ft	Depth and weight of sections					
	H beams				I beams	
	8-in 34-lb	6-in 23.8-lb	5-in 18.7-lb	4-in 13.6-lb	15-in 42-lb	12-in 31½-lb
2	130.0	91.0	71.5	52.0	162.2	120.4
3	130.0	91.0	71.5	52.0	162.2	120.4
4	130.0	91.0	71.5	52.0	162.2	120.4
5	130.0	91.0	71.5	50.7	162.2	120.4
6	130.0	91.0	71.5	45.7	153.9	109.9
7	130.0	91.0	66.0	40.6	140.1	98.9
8	130.0	86.7	60.5	35.6	126.2	87.9
9	130.0	80.9	55.0	30.5	112.3	76.9
10	125.8	75.1	49.5	26.7	98.5	65.9
11	119.4	69.3	44.0	24.2	86.0	59.9
12	113.0	63.5	38.5	21.7	79.0	54.4
13	106.6	57.7	35.8	19.2	72.1	48.9
14	100.2	51.9	33.0	16.6	65.2	43.4
15	93.8	47.6	30.3	14.1	58.2	37.9
16	87.3	44.7	27.5	....	51.3	32.4
17	80.9	41.8	24.8	....	44.4	26.9
18	74.5	38.9	22.0	....	37.4	....
19	69.0	36.0	19.3	....	....	....
20	65.8	33.1	16.5	....	....	....
21	62.6	30.2	....	....	....	....
22	59.4	27.3	....	....	....	....
23	56.2	24.4	....	....	....	....
24	53.0	21.5	....	....	....	....
25	49.8	....	....	....	....	....
26	46.6	....	....	....	....	....
27	43.4	....	....	....	....	....
28	40.2	....	....	....	....	....
29	37.0	....	....	....	....	....
30	33.7	....	....	....	....	....
31	30.5	....	....	....	....	....
Area, sq in	10.00	7.00	5.50	4.00	12.48	9.26
I_{1-1}, in^4	115.4	45.1	23.8	10.7	441.8	215.8
r_{1-1}, in	3.40	2.54	2.08	1.63	5.95	4.83
I_{2-2}, in^4	35.1	14.7	7.9	3.6	14.6	9.5
r_{2-2}, in	1.87	1.45	1.20	0.95	1.08	1.01
Weight, lb per lin ft	34	23.8	18.7	13.6	42	31½

Safe load-values above the upper heavy line are for ratios of l/r not over 6 between the heavy lines for ratios up to 120 l/r; and those below the lowe line are for ratios not over 200 l/r

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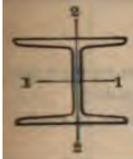
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbu

le XVII * (Continued). Safe Loads in Units of 1 000 Pounds for Steel-Beam Columns



Allowable fiber-stress in pounds per square inch: 13 000 for lengths of 60 radii or under

Reduced for lengths over 60 radii by Formula (13).

$$S = 19\,000 - 100\, l/r$$

Weights do not include details

fective ength, ft	Depth and weight of sections					
	I beams					
	9-in 21-lb	8-in 18-lb	7-in 15-lb	6-in 12¼-lb	5-in 9¾-lb	4-in 7½-lb
2	82.0	69.3	57.5	46.9	37.3	28.7
3	82.0	69.3	57.5	46.9	37.3	28.5
4	82.0	69.3	56.8	44.5	33.3	24.0
5	77.8	63.2	50.0	38.5	28.0	19.5
6	69.4	55.6	43.2	32.5	22.7	15.2
7	61.0	48.0	36.4	26.5	18.8	13.0
8	52.6	40.4	30.3	22.9	16.1	10.8
9	44.2	35.0	26.9	19.9	13.5	8.5
10	40.0	31.2	23.5	16.8	10.8	....
11	35.8	27.4	20.1	13.8	....	....
12	31.5	23.6	16.7	10.8	....	....
13	27.3	19.8	13.3	....	....	....
14	23.1	16.0	....	....	....	....
15	18.9	....	....	....	....	....
16	....	....	....	....	....	....
17	....	....	....	....	....	....
18	....	....	....	....	....	....
19	....	....	....	....	....	....
20	....	....	....	....	....	....
21	....	....	....	....	....	....
22	....	....	....	....	....	....
23	....	....	....	....	....	....
24	....	....	....	....	....	....
25	....	....	....	....	....	....
26	....	....	....	....	....	....
27	....	....	....	....	....	....
28	....	....	....	....	....	....
29	....	....	....	....	....	....
30	....	....	....	....	....	....
31	....	....	....	....	....	....
rea, sq in	6.31	5.33	4.42	3.61	2.87	2.21
$_{1-1}$, in^4.....	84.9	56.9	36.2	21.8	12.1	6.0
$_{1-1}$, in.....	3.67	3.27	2.86	2.46	2.05	1.64
$_{2-2}$, in^4.....	5.2	3.8	2.7	1.9	1.2	0.77
$_{2-2}$, in.....	0.90	0.84	0.78	0.72	0.65	0.59
Weight, per lin ft	21	18	15	12¼	9¾	7½

Safe load-values above the upper heavy line are for ratios of l/r not over 60; those etween the heavy lines for ratios up to 120 l/r; and those below the lower heavy ne are for ratios not over 200 l/r

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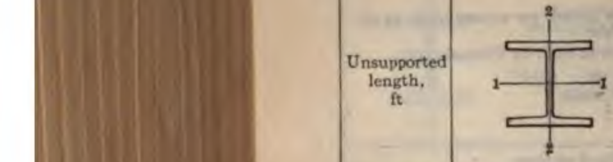
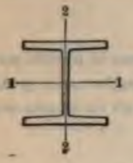
From Pocket Companion, *1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XVIII. Safe Loads in Tons of 2 000 Pounds for Bethlehe
Steel 8-Inch H Columns with Square Ends

Unsupported length, ft	 Allowable stress in pounds per squa 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55					
8	59.7	66.1	74.8	83.4	92.2	101.0
9	59.7	66.1	74.8	83.4	92.2	101.0
10	58.1	64.7	73.3	81.9	90.6	99.5
11	56.5	63.0	71.4	79.8	88.3	97.0
12	55.0	61.3	69.6	77.7	86.1	94.5
13	53.5	59.7	67.7	75.7	83.8	92.1
14	52.0	58.0	65.8	73.6	81.5	89.6
15	50.4	56.3	64.0	71.5	79.2	87.1
16	48.9	54.6	62.1	69.4	76.9	84.6
17	47.4	53.0	60.2	67.4	74.6	82.2
18	45.9	51.3	58.4	65.3	72.4	79.7
20	42.8	48.0	54.6	61.1	67.8	74.7
22	39.7	44.6	50.9	57.0	63.2	69.8
24	36.7	41.3	47.1	52.8	58.7	64.8
26	....	38.0	43.4	48.7	54.1	59.9
Area, sq in	9.17	10.17	11.50	12.83	14.18	15.53
I_{1-1}, in^4.....	105.7	121.5	139.5	158.3	177.7	197.8
r_{1-1}, in.....	3.40	3.46	3.48	3.51	3.54	3.57
I_{2-2}, in^4.....	35.8	41.1	47.2	53.4	59.8	66.3
r_{2-2}, in.....	1.98	2.01	2.03	2.04	2.05	2.07
Weight of section, lb per lin ft	32.0	34.5	39.0	43.5	48.0	53.0

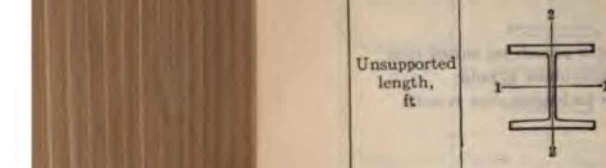
Loads below the heavy line are for lengths greater than 125 radi

XVIII (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 8-Inch H Columns with Square Ends

rted n,	 Allowable stress in pounds per square inch: 13 000 for lengths under 55 radii; 16 000 − 55 *l*/*r* for lengths over 55 radii						
3	118.8	127.8	136.8	146.0	154.6	163.8	173.2
9	118.8	127.8	136.8	146.0	154.6	163.8	173.2
0	117.3	126.5	135.6	144.9	153.6	163.1	172.6
1	114.4	123.5	132.4	141.4	149.9	159.3	168.6
2	111.5	120.4	129.1	137.9	146.2	155.4	164.5
3	108.7	117.3	125.8	134.4	142.6	151.6	160.5
4	105.8	114.2	122.5	131.0	138.9	147.7	156.4
5	102.9	111.2	119.2	127.5	135.2	143.9	152.4
6	100.0	108.1	116.0	124.0	131.6	140.0	148.3
7	97.1	105.0	112.7	120.5	127.9	136.1	144.3
8	94.2	101.9	109.4	117.0	124.2	132.3	140.2
0	88.5	95.8	102.9	110.1	116.9	124.6	132.1
2	82.7	89.6	96.3	103.1	109.6	116.9	124.0
4	76.9	83.5	89.8	96.2	102.2	109.2	115.9
6	71.2	77.3	83.2	89.2	94.9	101.5	107.8
sq in	18.27	19.66	21.05	22.46	23.78	25.20	26.64
⁴.....	240.2	262.5	285.6	309.5	333.5	359.0	385.3
......	3.63	3.65	3.68	3.71	3.75	3.77	3.80
⁴.....	80.0	87.1	94.4	101.9	109.2	117.2	125.1
......	2.09	2.11	2.12	2.13	2.14	2.16	2.17
ht ion, in ft	62.0	67.0	71.5	76.5	81.0	85.5	90.5

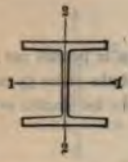
Loads below the heavy line are for lengths greater than 125 radii

Table XIX. Safe Loads in Tons of 2 000 Pounds for Bethlehem
Steel 10-Inch H Columns with Square Ends

Unsupported length, ft	 Allowable stress in pounds per 13000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55					
10	93.5	103.4	114.2	125.0	135.9	146.8
11	93.5	103.4	114.2	125.0	135.9	146.8
12	92.1	102.2	113.1	123.9	134.9	145.9
13	90.2	100.1	110.8	121.4	132.2	143.0
14	88.3	98.0	108.5	118.9	129.5	140.1
15	86.3	95.9	106.2	116.4	126.9	137.2
16	84.5	93.8	103.9	113.9	124.2	134.3
18	80.7	89.6	99.3	108.9	118.8	128.5
20	76.9	85.4	94.7	103.9	113.4	122.7
22	73.1	81.3	90.1	98.9	108.0	116.9
24	69.3	77.1	85.6	93.9	102.6	111.1
26	65.4	72.9	81.0	88.9	97.2	105.3
28	61.6	68.7	76.4	83.9	91.8	99.5
30	57.8	64.5	71.8	78.9	86.4	93.7
32	54.0	60.3	67.2	73.9	80.1	87.9
Area, sq in	14.37	15.91	17.57	19.23	20.91	22.59
I_{1-1}, in^4.....	263.5	296.8	331.9	368.0	405.2	443.6
r_{1-1}, in.....	4.28	4.32	4.35	4.37	4.40	4.43
I_{2-2}, in^4.....	89.1	100.4	112.2	124.2	136.5	149.1
r_{2-2}, in.....	2.49	2.51	2.53	2.54	2.56	2.57
Weight of section, lb per lin ft	49.0	54.0	59.5	65.5	71.0	77.0

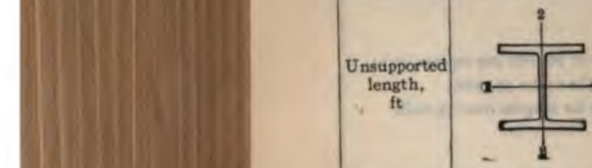
Loads below the heavy line are for lengths greater than 125 radii

ble XIX (Continued). **Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 10-Inch H Columns with Square Ends**

upported length, ft	 Allowable stress in pounds per square inch: 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 radii						
10	168.9	180.1	190.6	201.9	213.2	224.6	236.1
11	168.9	180.1	190.6	201.9	213.2	224.6	236.1
12	168.3	179.6	190.2	201.9	213.2	224.6	236.1
13	165.0	176.1	186.6	198.0	209.3	220.7	232.2
14	161.7	172.6	182.9	194.1	205.2	216.4	227.7
15	158.4	169.1	179.2	190.3	201.2	212.1	223.2
16	155.1	165.6	175.5	186.4	197.0	207.8	218.7
18	148.5	158.6	168.1	178.6	188.9	199.2	209.8
20	142.0	151.6	160.7	170.8	180.7	190.7	200.8
22	135.4	144.6	153.3	163.1	172.5	182.1	191.8
24	128.8	137.6	145.9	155.3	164.4	173.5	182.8
26	122.2	130.6	138.5	147.5	156.2	165.0	173.8
28	115.6	123.6	131.2	139.8	148.0	156.4	164.9
30	109.0	116.6	123.8	132.0	139.9	147.8	155.9
32	102.4	109.6	116.4	124.2	131.7	139.2	146.9
rea, sq in	25.99	27.71	29.32	31.06	32.80	34.55	36.32
$_{1-1}$, in^4.....	523.5	565.2	607.0	651.0	696.2	742.7	790.4
$_{1-1}$, in.....	4.49	4.52	4.55	4.58	4.61	4.64	4.67
$_{2-2}$, in^3.....	175.1	188.6	201.7	215.6	229.9	244.4	259.3
$_{2-2}$, in.....	2.60	2.61	2.62	2.64	2.65	2.66	2.67
eight of section, per lin ft	88.5	94.0	99.5	105.5	111.5	117.5	123.5

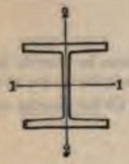
Loads below the heavy line are for lengths greater than 125 radii

Table XX. Safe Loads in Tons of 2 000 Pounds for Bethlehem Steel 12-Inch H Columns with Square Ends

Unsupported length, ft	 Allowable stress in pounds per square 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55					
10	123.5	136.2	149.1	162.0	175.0	188.0	201.1
12	123.5	136.2	149.1	162.0	175.0	188.0	201.1
14	122.5	135.4	148.3	161.4	174.5	187.7	201.0
16	118.3	130.8	143.3	155.9	168.6	181.5	194.3
18	114.1	126.2	138.3	150.5	162.8	175.2	187.7
20	109.9	121.6	133.2	145.1	156.9	169.0	181.0
22	105.7	117.0	128.2	139.7	151.1	162.8	174.4
24	101.5	112.4	123.2	134.2	145.2	156.5	167.7
26	97.3	107.8	118.1	128.8	139.4	150.3	161.1
28	93.1	103.1	113.1	123.4	133.5	144.0	154.4
30	88.9	98.5	108.1	117.9	127.7	137.8	147.8
32	84.7	93.9	103.0	112.5	121.9	131.6	141.1
34	80.5	89.3	98.0	107.1	116.0	125.3	134.4
36	76.3	84.7	93.0	101.7	110.2	119.1	127.8
38	......	......	......	......	104.3	112.8	121.1
Area, sq in	19.00	20.96	22.94	24.92	26.92	28.92	30.91
I_{1-1}, in^4.....	499.0	556.6	615.6	676.1	738.1	801.7	866.8
r_{1-1}, in.....	5.13	5.15	5.18	5.21	5.24	5.27	5.30
I_{2-2}, in^4.....	168.6	188.2	208.1	228.5	249.2	270.1	291.7
r_{2-2}, in.....	2.98	3.00	3.01	3.03	3.04	3.06	3.07
Weight of section, lb per lin ft	64.5	71.5	78.0	84.5	91.5	98.5	105.0

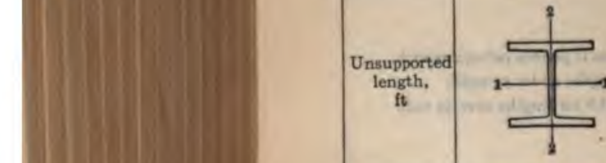
Loads below the heavy line are for lengths greater than 125 radii

able XX (Continued). **Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 12-Inch H Columns with Square Ends**

upported ength, ft	 Allowable stress in pounds per square inch: 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 radii						
10	226.7	239.9	253.3	266.7	280.2	293.7	307.3
12	226.7	239.9	253.3	266.7	280.2	293.7	307.3
14	226.7	239.9	253.3	266.7	280.2	293.7	307.3
16	219.6	232.6	246.0	259.3	272.6	286.0	299.7
18	212.1	224.8	237.8	250.6	263.5	276.6	289.9
20	204.7	217.0	229.6	242.0	254.5	267.1	280.1
22	197.3	209.1	221.4	233.4	245.5	257.7	270.3
24	189.9	201.3	213.2	224.8	236.4	248.3	260.5
26	182.5	193.5	204.9	216.1	227.4	238.8	250.7
28	175.0	185.6	196.7	207.5	218.4	229.4	240.9
30	167.6	177.8	188.5	198.9	209.3	219.9	231.0
32	160.2	170.0	180.3	190.3	200.3	210.5	221.2
34	152.8	162.1	172.1	181.6	191.3	201.1	211.4
36	145.3	154.3	163.9	173.0	182.3	191.6	201.6
38	137.9	146.5	155.6	164.4	173.2	182.2	191.8
Area, sq in	34.87	36.91	38.97	41.03	43.10	45.19	47.28
I_{1-1}, in^4.....	1 000.0	1 069.8	1 141.3	1 214.5	1 289.4	1 366.0	1 444.3
r_{1-1}, in.....	5.36	5.38	5.41	5.44	5.47	5.50	5.63
I_{2-2}, in^4.....	335.0	357.7	380.7	404.1	428.0	452.2	477.0
r_{2-2}, in.....	3.10	3.11	3.13	3.14	3.15	3.16	3.18
Weight f section, b per lin ft	118.5	125.5	132.5	139.5	146.5	153.5	161.0

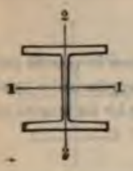
Loads below the heavy line are for lengths greater than 125 radii

Table XXI. Safe Loads in Tons of 2 000 Pounds for Bethlehem
Steel 14-Inch H Columns with Square Ends

Unsupported length, ft	 Allowable stress in pounds per square 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 radii				
10	159.0	173.9	188.9	204.0	219.1	234.3
12	159.0	173.9	188.9	204.0	219.1	234.3
14	159.0	173.9	188.9	204.0	219.1	234.3
16	158.5	173.9	188.6	204.0	219.1	234.3
18	153.8	168.5	183.2	198.1	212.9	228.0
20	149.2	163.4	177.7	192.2	206.6	221.3
22	144.5	158.4	172.2	186.3	200.3	214.6
24	139.9	153.3	166.7	180.4	194.0	207.9
26	135.2	148.2	161.2	174.6	187.7	201.2
28	130.5	143.2	155.8	168.7	181.4	194.5
30	125.9	138.1	150.3	162.8	175.1	187.8
32	121.2	133.1	144.8	156.9	168.8	181.1
36	111.9	122.9	133.8	145.1	156.2	167.7
40	102.6	112.8	122.9	133.4	143.6	154.3
44	....	....	111.9	121.6	131.0	140.9
Area, sq in	24.46	26.76	29.06	31.38	33.70	36.04
I_{1-1}, in^4.....	884.9	976.8	1 070.6	1 166.6	1 264.5	1 364.6
r_{1-1}, in......	6.01	6.04	6.07	6.10	6.13	6.16
I_{2-2}, in^4.....	294.5	325.4	356.9	387.8	420.3	453.4
r_{2-2}, in.....	3.47	3.49	3.50	3.52	3.53	3.55
Weight of section, lb per lin ft	83.5	91.0	99.0	106.5	114.5	122.5

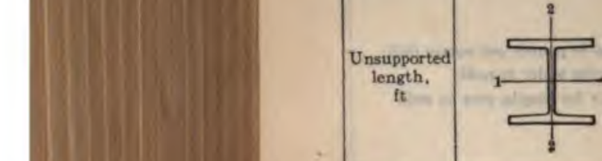
Loads below the heavy line are for lengths greater than 125 radii

able XXI (Continued). **Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 14-Inch H Columns with Square Ends**

supported length, ft	 Allowable stress in pounds per square inch: 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 radii					
10	263.8	279.2	294.7	310.1	325.7	341.3
12	263.8	279.2	294.7	310.1	325.7	341.3
14	263.8	279.2	294.7	310.1	325.7	341.3
16	263.8	279.2	294.7	310.1	325.7	341.3
18	257.4	272.5	288.1	303.4	319.0	334.6
20	249.9	264.6	279.8	294.7	309.9	325.1
22	242.4	256.7	271.5	286.0	300.8	315.6
24	234.9	248.9	263.2	277.3	291.7	306.1
26	227.4	241.0	254.9	268.6	282.6	296.6
28	220.0	233.1	246.6	259.9	273.5	287.1
30	212.5	225.2	238.3	251.2	264.4	277.6
32	205.0	217.3	230.0	242.5	255.3	268.1
36	190.0	201.5	213.5	225.1	237.1	249.1
40	175.1	185.7	196.9	207.7	218.9	230.1
44	160.1	170.0	180.3	190.3	200.7	211.1
Area, sq in	40.59	42.95	45.33	47.71	50.11	52.51
I_{1-1}, in^4……	1 568.4	1 674.7	1 783.3	1 894.0	2 007.0	2 122.3
r_{1-1}, in……	6.21	6.24	6.27	6.30	6.33	6.36
I_{2-2}, in^4……	519.7	554.4	589.5	626.1	662.3	699.0
r_{2-2}, in……	3.58	3.59	3.61	3.62	3.64	3.65
Weight of section, b per lin ft	138.0	146.0	154.0	162.0	170.5	178.5

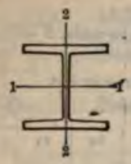
Loads below the heavy line are for lengths greater than 125 radii

Table XXI (Continued). Safe Loads in Tons of 2 000 Pounds for B lehem Rolled-Steel 14-Inch H Columns with Square Ends

Unsupported length, ft	 Allowable stress in pounds per squar 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 ra					
10	357.0	372.8	388.6	403.5	419.4	435.4
12	357.0	372.8	388.6	403.5	419.4	435.4
14	357.0	372.8	388.6	403.5	419.4	435.4
16	357.0	372.8	388.6	403.5	419.4	435.4
18	350.3	366.1	381.9	396.9	412.9	429.0
20	340.4	355.8	371.2	385.8	410.5	417.2
22	330.5	345.5	360.5	374.8	390.0	405.3
24	320.6	335.2	349.8	363.7	378.5	393.4
26	310.7	324.9	339.1	352.6	367.1	381.6
28	300.8	314.6	328.4	341.6	355.6	369.7
30	290.9	304.3	317.7	330.5	344.1	357.8
32	281.0	294.0	307.0	319.4	332.6	345.9
36	261.2	273.4	285.6	297.3	309.7	322.1
40	241.4	252.8	264.2	275.1	286.8	298.5
44	221.6	232.2	242.8	253.0	263.8	274.8
Area, sq in	54.92	57.35	59.78	62.07	64.52	66.98
I_{1-1}, in⁴	2 239.8	2 359.7	2 481.9	2 603.3	2 730.2	2 859.6
r_{1-1}, in	6.39	6.41	6.44	6.48	6.51	6.53
I_{2-2}, in⁴	736.3	744.2	812.6	849.8	889.3	929.4
r_{2-2}, in	3.66	3.67	3.69	3.70	3.71	3.73
Weight of section, lb per lin ft	186.5	195.0	203.5	211.0	219.5	227.5

Loads below the heavy line are for lengths greater than 125 radii

able XXI (Continued). **Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 14-Inch H Columns with Square Ends**

supported length, ft	 Allowable stress in pounds per square inch: 13 000 for lengths under 55 radii; 16 000 − 55 l/r for lengths over 55 radii					
10	467.6	483.8	500.0	516.4	532.8	549.3
12	467.6	483.8	500.0	516.4	532.8	549.3
14	467.6	483.8	500.0	516.4	532.8	549.3
16	467.6	483.8	500.0	516.4	532.8	549.3
18	461.5	477.9	494.4	510.9	527.6	544.3
20	448.9	464.8	480.9	497.0	513.3	529.6
22	436.2	451.8	467.4	483.2	499.1	515.0
24	423.6	438.7	454.0	469.3	484.8	500.4
26	410.9	425.7	440.5	455.5	470.6	485.7
28	398.2	412.6	427.1	441.6	456.3	471.1
30	385.6	399.6	413.6	427.8	442.1	456.4
32	372.9	386.5	400.2	413.9	427.8	441.8
36	347.6	360.4	373.3	386.3	399.4	412.5
40	322.2	334.3	346.4	358.6	370.9	383.3
44	296.9	308.1	319.5	330.9	342.4	354.0
Area, sq in	71.94	74.43	76.93	79.44	81.97	84.50
I_{1-1}, in^4.....	3 125.8	3 262.7	3 402.1	3 544.1	3 688.8	3 836.1
r_{1-1}, in.....	6.59	6.62	6.65	6.68	6.71	6.74
I_{2-2}, in^4.....	1 011.3	1 053.2	1 095.6	1 138.7	1 182.4	1 226.7
r_{2-2}, in.....	3.75	3.76	3.77	3.79	3.80	3.81
Weight of section, lb per lin ft	244.5	253.0	261.5	270.0	278.5	287.5

Loads below the heavy line are for lengths greater than 125 radii

Table XXII. Safe Loads in Tons of 2 000
Columns

Factor of safety
Calculated by the New York building cod

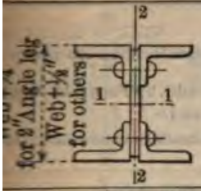
Outside diameter, in	Length of col					
	6	7	8	9	10	11
3	6	6	5	....	....	...
3½	9	9	8	8	7	...
4	13	13	12	12	11	10
4½	14	14	13	13	12	11
5	20	20	19	19	18	18
6	28	28	27	27	26	26

Table XXIII. Safe Loads in Tons of 2
Lally Column

Factor of safety
These loads can be greatly increased b
Calculated by the New York building cod

Outside diameter, in	Weight per linear foot, lb	Length of co		
		6	8	10
2⅞	10	8	7	6
3½	15	12	11	10
4	20	16	15	14
4½	24	20	18	17
5	29	27	26	24
5½	36	32	31	29
6⅝	49	45	43	41
7⅝	64	58	56	54
8⅝	81	74	72	69
9⅝	100	93	89	87
10¾	123	111	109	107
11¾	146	131	128	124
12¾	169	150	146	144

Table XXIV.* Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Web-plate 6"×1/4"			Web-plate 8"×1/4"			
	4 angles 2½"×2"×1/4"	4 angles 3"×2"×1/4"	4 angles 3"×2½"×1/4"	4 angles 3"×2½"×1/4"	4 angles 3"×2½"×5/16"	4 angles 3½"×2½"×1/4"	4 angles 3½"×2½"×5/16"
6	69	81	88	94	110	101	119
7	63	78	82	86	103	101	119
8	56	72	76	79	95	96	115
9	49	66	69	72	87	89	107
10	43	60	63	65	78	83	100
11	38	54	56	57	70	76	92
12	35	49	50	50	62	70	85
13	32	43	45	47	56	63	78
14	28	40	42	43	52	57	70
15	25	37	39	39	48	52	63
16	22	34	35	36	44	49	60
17	18	32	32	32	40	46	56
18	....	29	29	28	36	43	52
19	....	26	26	25	32	39	49
20	....	23	22	....	28	36	45
21	....	20	....	....	....	33	41
22	....	....	....	....	....	30	38
23	....	....	....	....	....	27	34
24	....	....	....	....	....	23	30
25	....	....	....	....	....	....	....
26	....	....	....	....	....	....	....
27	....	....	....	....	....	....	....
28	....	....	....	....	....	....	....
29	....	....	....	....	....	....	....
30	....	....	....	....	....	....	....
Area, sq in	5.74	6.26	6.74	7.24	8.48	7.76	9.12
I_{1-1}, in^4.....	34.3	39.1	42.6	81.2	96.9	90.1	107
r_{1-1}, in.....	2.45	2.50	2.51	3.35	3.38	3.41	3.43
I_{2-2}, in^4.....	6.2	10.3	10.3	10.3	12.9	16.0	20.2
r_{2-2}, in.....	1.04	1.28	1.24	1.19	1.23	1.44	1.49
Weight, lb per lin ft..	19.6	21.5	23.1	24.8	29.2	26.4	31.2

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those *below the lower* heavy line are for ratios not over 200 l/r

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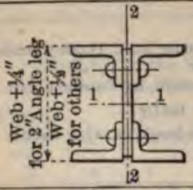
* *From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds
and-Angle Columns



Allowable fiber-stress per square inch: 13 000
for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

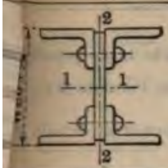
Weights do not include rivet-heads or other

Effective length, ft	Web-plate 8"×5/16"				Web-plate 8"×	
	4 angles 3½"×2½"×5/16"	4 angles 3½"×2½"×3/8"	4 angles 4"×3"×5/16"	4 angles 4"×3"×3/8"	4 angles 4"×3"×3/8"	4 angles 4"×3"×7/16"
6	125	142	141	161	168	188
7	125	142	141	161	168	188
8	120	138	141	161	168	188
9	112	130	136	158	163	185
10	104	121	128	149	154	175
11	96	112	121	140	145	165
12	89	104	113	131	136	155
13	81	95	105	123	127	145
14	73	86	97	114	118	135
15	66	77	89	105	109	124
16	62	73	81	97	100	114
17	58	68	75	88	90	104
18	54	64	71	83	86	98
19	50	60	67	79	81	93
20	47	55	63	74	77	88
21	43	51	59	70	72	83
22	39	47	55	66	68	78
23	35	42	51	61	63	73
24	31	38	48	57	59	68
25	....	34	44	53	54	63
26	....	....	40	48	49	58
27	....	....	36	44	45	53
28	....	....	....	39	40	48
29	....	....	....	....	....	....
30	....	....	....	....	....	....
Area, sq in	9.62	10.94	10.86	12.42	12.92	14.48
I_{1-1}, in⁴.....	110	127	122	141	143	161
r_{1-1}, in.....	3.38	3.40	3.35	3.36	3.33	3.34
I_{2-2}, in⁴.....	20.7	24.9	30.3	36.3	37.2	43.5
r_{2-2}, in.....	1.47	1.51	1.67	1.71	1.70	1.73
Weight, lb per lin ft..	32.9	37.3	37.3	42.5	44.2	49.4

The safe load-values above the upper heavy line are for ratios of l/r not those between the heavy lines are for ratios up to 120 l/r; and those below t heavy line are for ratios not over 200 l/r

* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

ble XXIV[*] (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

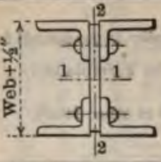
Weights do not include rivet-heads or other details

Effective length, ft	Web-plate 10″×¼″			Web-plate 10″×5/16″			Web-plate 10″×3/8″		
	4 angles 3″×2½″×¼″	4 angles 3½″×2½″×¼″	4 angles 3½″×2½″×5/16″	4 angles 3½″×2½″×5/16″	4 angles 4″×3″×5/16″	4 angles 4″×3″×3/8″	4 angles 4″×3″×3/8″	4 angles 4″×3″×7/16	4 angles 5″×3½″×3/8″
6	99	107	125	133	149	170	178	198	207
7	91	107	125	133	149	170	178	198	207
8	82	100	119	125	149	170	178	198	207
9	74	93	111	117	142	164	170	192	207
10	66	86	103	108	133	154	160	181	207
11	58	79	95	99	125	145	150	170	203
12	52	71	87	91	116	135	140	160	194
13	48	64	79	82	108	126	130	149	185
14	44	57	71	73	99	117	121	138	175
15	40	54	65	68	91	107	111	127	166
16	36	50	61	64	82	98	101	116	157
17	32	47	57	60	77	90	93	106	148
18	28	43	53	55	73	85	88	101	139
19	24	40	49	51	69	81	83	95	130
20	.....	36	45	47	64	76	78	90	121
21	.....	33	41	42	60	71	73	84	112
22	.....	29	37	38	56	67	68	79	107
23	.....	25	34	34	51	62	63	74	103
24	.....	.....	30	.....	47	57	58	68	98
25	.....	.....	.....	.....	43	52	53	63	93
26	.....	.....	.....	.....	39	48	48	57	89
27	.....	.....	.....	.....	34	43	43	52	84
28	.....	.....	.....	.....	.....	.....	.....	47	80
29	.....	.....	.....	.....	.....	.....	.....	.....	75
30	.....	.....	.....	.....	.....	.....	.....	.....	71
rea, sq in	7.74	8.26	9.62	10.25	11.49	13.05	13.67	15.23	15.95
-1, in^4.....	134	148	176	181	201	232	237	267	279
-1, in.....	4.16	4.23	4.28	4.20	4.18	4.22	4.17	4.19	4.18
-2, in^4.....	10.3	16.0	20.2	26.7	30.3	36.3	37.2	43.5	70.6
-2, in.....	1.15	1.39	1.45	1.42	1.62	1.67	1.65	1.69	2.10
Weight, per lin ft ..	26.5	28.1	32.9	35.0	39.4	44.6	46.8	52.0	54.4

The safe load-values above the upper heavy line are for ratios of l/r not over 60; hose between the heavy lines are for ratios up to 120 l/r; and those below the lower eavy line are for ratios not over 200 l/r

[*] *From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for P and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pou lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (1)

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other deta

Effective length, ft	Web-plate 10"×⅜"				Web-plate 10"×½"		
	4 angles 5"×3½"×5/16	4 angles 6"×4"×⅜"	4 angles 6"×4"×7/16"	4 angles 6"×4"×½"	4 angles 6"×4"×½"	4 angles 6"×4"×9/16	4 angles 6"×4"×⅝"
6	232	236	266	296	312	341	370
7	232	236	266	296	312	341	370
8	232	236	266	296	312	341	370
9	232	236	266	296	312	341	370
10	232	236	266	296	312	341	370
11	230	236	266	296	312	341	370
12	220	236	266	296	312	341	370
13	210	235	266	296	312	341	370
14	200	226	257	288	302	333	363
15	190	218	248	278	291	321	350
16	180	209	238	267	280	309	337
17	170	201	229	257	269	297	325
18	160	192	220	247	258	285	312
19	150	184	210	237	247	274	299
20	140	175	201	226	236	262	287
21	130	167	191	216	225	250	274
22	123	158	182	206	214	238	261
23	118	150	172	195	203	226	249
24	113	141	163	185	192	214	236
25	108	132	154	175	181	203	223
26	103	126	144	164	170	191	210
27	98	121	139	157	164	181	198
28	93	117	134	152	158	175	192
29	88	113	130	146	153	169	186
30	83	109	125	141	147	164	179
Area, sq in	17.87	18.19	20.47	22.75	24.00	26.24	28.44
I_{1-1}, in^4.....	315	319	361	401	412	451	489
r_{1-1}, in.....	4.20	4.19	4.20	4.20	4.14	4.15	4.15
I_{2-2}, in^4.....	82.3	119	139	160	165	186	206
r_{2-2}, in.....	2.15	2.56	2.61	2.65	2.62	2.66	2.69
Weight, lb per lin ft..	60.8	62.0	70.0	77.6	81.8	89.4	97.0

The safe load-values above the upper heavy line are for ratios of l/r not o those between the heavy lines are for ratios up to 120 l/r; and those below th heavy line are for ratios not over 200 l/r

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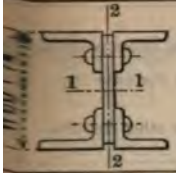
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbu

ble XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch, 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Web-plate 12″×¼″			Web-plate 12″×5/16″		Web-plate 12″×3/8″			
	4 angles 3½″×2½″×¼″	4 angles 3½″×2½″×5/16″	4 angles 4″×3″×5/16″	4 angles 4″×3″×5/16″	4 angles 4″×3″×3/8″	4 angles 4″×3″×3/8″	4 angles 5″×3½″×3/8″	4 angles 5″×3½″×7/16″	4 angles 5″×3½″×½″
6	114	132	148	157	178	187	217	242	266
7	112	132	148	157	178	187	217	242	266
8	104	123	148	157	178	187	217	242	266
9	96	115	140	147	169	177	217	242	266
10	89	106	131	138	159	167	217	242	266
11	81	98	123	129	149	156	210	237	264
12	73	89	114	120	139	145	201	226	252
13	65	80	106	111	129	134	191	215	241
14	59	72	97	101	119	124	181	205	229
15	55	67	89	92	109	113	171	194	218
16	52	63	80	84	99	102	162	184	206
17	48	58	76	79	92	96	152	173	195
18	44	54	71	75	87	91	142	162	184
19	40	50	67	70	82	85	132	152	172
20	36	45	63	65	77	80	123	141	161
21	32	41	59	61	72	75	115	130	149
22	28	37	55	56	67	69	110	125	141
23	......	33	50	52	62	64	105	120	135
24	......	......	46	47	57	58	100	114	129
25	......	......	42	42	52	53	95	109	123
26	......	......	38	38	47	48	91	104	118
27	......	......	......	......	42	......	86	98	112
28	......	......	......	......	......	......	81	93	106
29	......	......	......	......	......	......	76	88	101
30	......	......	......	......	......	......	71	82	95
Area, sq in	8.76	10.12	11.36	12.11	13.67	14.42	16.70	18.62	20.50
I_{1-1}, in.^{4}	222	264	295	304	350	359	421	476	526
r_{1-1}, in	5.04	5.11	5.09	5.01	5.06	4.99	5.02	5.05	5.07
I_{2-2}, in.^{4}	16.0	20.2	29.6	30.3	36.3	37.3	70.6	82.3	94.6
r_{2-2}, in	1.35	1.41	1.61	1.58	1.63	1.61	2.06	2.10	2.15
Weight, lb per lin ft ..	29.8	34.6	39.0	41.6	46.8	49.3	56.9	63.3	69.7

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not *over 200 l/r*

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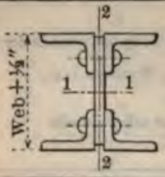
* *From Pocket Companion, 1915 Edition, Carnegie Steel Company*, Pittsburgh, Pa.

Table XXIV * **(Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns**



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

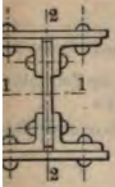
Weights do not include rivet-heads or other details

Effective length, ft	Web-plate 12″×⅜″		Web-plate 12″×½″					Web-plate
								12″×⅝″
	4 angles 6″×4″×7/16″	4 angles 6″×4″×½″	4 angles 6″×4″×½″	4 angles 6″×4″×9/16″	4 angles 6″×4″×⅝″	4 angles 6″×4″×11/16″	4 angles 6″×4″×¾″	4 angles 6″×4″×¾″
6	276	305	325	354	383	411	439	458
7	276	305	325	354	383	411	439	458
8	276	305	325	354	383	411	439	458
9	276	305	325	354	383	411	439	458
10	276	305	325	354	383	411	439	458
11	276	305	325	354	383	411	439	458
12	276	305	325	354	383	411	439	458
13	274	305	323	354	383	411	439	458
14	264	295	312	342	373	403	433	451
15	254	284	300	330	359	389	418	435
16	244	274	288	317	346	375	403	419
17	234	263	277	305	333	361	388	404
18	224	252	265	292	319	347	373	388
19	214	241	253	280	306	333	358	372
20	204	230	242	267	293	318	344	357
21	194	220	230	255	279	304	329	341
22	184	209	218	242	266	290	314	325
23	174	198	207	230	253	276	299	310
24	164	187	195	217	239	262	284	294
25	155	176	183	204	226	248	269	278
26	147	166	173	192	213	234	254	262
27	142	160	167	185	203	220	239	247
28	137	154	162	179	196	213	230	239
29	132	149	156	173	189	206	223	231
30	127	143	150	166	183	199	215	223
Area, sq in	21.22	23.50	25.00	27.24	29.44	31.60	33.76	35.26
I_{1-1}, in^4	544	605	623	683	741	794	849	867
r_{1-1}, in	5.06	5.07	4.99	5.01	5.02	5.01	5.01	4.96
I_{2-2}, in^4	139	160	165	186	206	228	249	257
r_{2-2}, in	2.56	2.61	2.57	2.61	2.65	2.69	2.72	2.70
Weight, lb per lin ft	72.5	80.1	85.2	92.8	100.4	107.6	114.8	119.9

The safe load-values above the upper heavy line are for ratios of l/r not over those between the heavy lines are for ratios up to 120 l/r; and those below the heavy line are for ratios not over 200 l/r

* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

XIV * (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

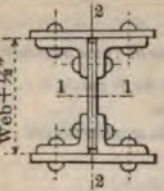
Weights do not include rivet-heads or other details

	Web-plate 12"×3/8"				Web-plate 12"×1/2"	
ve n.	4 angles 6"×4"×3/8" 2 plates 14"×3/8"	4 angles 6"×4"×3/8" 2 plates 14"×1/2"	4 angles 6"×4"×7/16" 2 plates 14"×1/2"	4 angles 6"×4"×1/2" 2 plates 14"×1/2"	4 angles 6"×4"×1/2" 2 plates 14"×1/2"	4 angles 6"×4"×1/2" 2 plates 14"×5/8"
	383	428	458	487	507	553
	383	428	458	487	507	553
	383	428	458	487	507	553
	383	428	458	487	507	553
	383	428	458	487	507	553
	379	428	458	487	506	553
	368	419	447	475	491	542
	357	407	434	461	476	526
	346	395	421	447	461	510
	334	383	407	433	447	495
	323	370	394	419	432	479
	312	358	381	405	417	463
	301	346	368	391	403	448
	289	334	355	377	388	432
	278	322	342	363	373	416
	267	310	329	349	358	401
	256	297	316	335	344	385
	244	285	303	321	329	369
	233	273	290	307	314	354
	222	261	277	293	299	338
	211	249	264	279	285	323
	203	237	250	265	272	307
	197	228	242	257	264	294
	191	221	235	250	257	287
	186	215	229	243	249	279
in	29.44	32.94	35.22	37.50	39.00	42.50
....	916	1 073	1 136	1 197	1 215	1 377
....	5.58	5.71	5.68	5.65	5.58	5.69
....	291	348	368	388	394	451
....	3.14	3.25	3.23	3.22	3.18	3.26
, ft..	100.2	112.1	120.1	127.7	132.8	144.7

fe load-values above the upper heavy line are for ratios of l/r not over 60; tween the heavy lines are for ratios up to 120 l/r; and those below the lower ne are for ratios not over 200 l/r

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for
and-Angle Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other

Effective length, ft	Web-plate 12"×1/2"		Web-plate 12"×5/8"		
	4 angles 6"×4"×9/16" 2 plates 14"×5/8"	4 angles 6"×4"×5/8" 2 plates 14"×5/8"	4 angles 6"×4"×5/8" 2 plates 14"×5/8"	4 angles 6"×4"×5/8" 2 plates 14"×3/4"	4 angles 6"×4"×5/8" 2 plates 14"×7/8"
11	582	610	630	675	721
12	582	610	630	675	721
13	582	610	630	675	721
14	582	610	630	675	721
15	582	610	630	675	721
16	582	610	630	675	721
17	569	596	613	663	714
18	553	579	594	644	694
19	536	562	576	625	674
20	520	544	558	606	654
21	503	527	540	587	634
22	487	509	522	568	614
23	470	492	504	548	594
24	454	475	486	529	574
25	437	457	468	510	554
26	421	440	450	491	534
27	404	422	431	472	514
28	388	405	413	453	494
29	371	388	395	434	474
30	354	370	377	415	454
31	338	353	359	396	434
32	321	336	341	377	414
33	309	323	331	361	394
34	301	315	322	351	381
35	293	306	313	342	371
Area, sq in	44.74	46.94	48.44	51.94	55.44
I_{1-1}, in^4.....	1 437	1 495	1 513	1 682	1 856
r_{1-1}, in......	5.67	5.64	5.59	5.69	5.79
I_{2-2}, in^4....	472	492	499	556	613
r_{2-2}, in.....	3.25	3.24	3.21	3.27	3.33
Weight, lb per lin ft..	152.3	159.9	165.0	176.9	188.8

The safe load-values above the upper heavy line are for ratios of l/r not
those between the heavy lines are for ratios up to 120 l/r; and those below the
heavy line are for ratios not over 200 l/r

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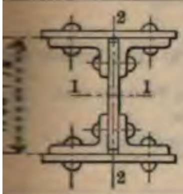
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

le XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

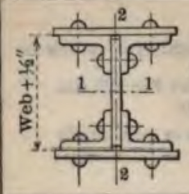
Weights do not include rivet-heads or other details

ffective ength, ft	Web-plate 12″×5/8″					
	4 angles 6″×4″×5/8″ 2 plates 14″×1 1/8″	4 angles 6″×4″×5/8″ 2 plates 14″×1 1/4″	4 angles 6″×4″×5/8″ 2 plates 14″×1 3/8″	4 angles 6″×4″×5/8″ 2 plates 14″×1 1/2″	4 angles 6″×4″×5/8″ 2 plates 14″×1 5/8″	4 angles 6″×4″×5/8″ 2 plates 14″×1 3/4″
11	812	857	903	948	994	1 039
12	812	857	903	948	994	1 039
13	812	857	903	948	994	1 039
14	812	857	903	948	994	1 039
15	812	857	903	948	994	1 039
16	812	857	903	948	994	1 039
17	812	857	903	948	994	1 039
18	791	840	888	937	986	1 034
19	769	817	864	912	960	1 007
20	747	794	840	887	934	980
21	725	771	817	862	908	953
22	703	748	793	837	882	926
23	681	725	769	812	856	899
24	659	702	745	787	830	872
25	637	679	721	762	805	845
26	615	657	697	738	779	818
27	593	634	673	713	753	791
28	571	611	649	688	727	764
29	549	588	625	663	701	737
30	527	565	601	638	675	710
31	505	542	577	613	649	684
32	483	519	553	588	623	657
33	461	496	529	563	597	630
34	439	473	505	538	571	603
35	427	456	484	513	545	576
Area, sq in	62.44	65.94	69.44	72.94	76.44	79.94
I_{1-1}, in⁴	2 224	2 418	2 618	2 825	3 038	3 259
r_{1-1}, in	5.97	6.06	6.14	6.22	6.30	6.38
I_{2-2}, in⁴	728	785	842	899	956	1014
r_{2-2}, in	3.41	3.45	3.48	3.51	3.54	3.56
Weight, lb per lin ft	212.6	224.5	236.4	248.3	260.2	272.1

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those *below the lower* heavy line are for ratios not over 200 l/r

* *From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds
and-Angle Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

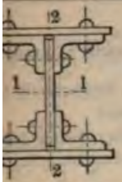
Weights do not include rivet-heads or other

Effective length, ft	Web-plate 12"×5/8": 4 angles 6"×4"×5/8", 2 plates 14"×1 7/8"	Web-plate 12"×5/8": 4 angles 6"×4"×5/8", 2 plates 14"×2"	Web-plate 14"×3/8": 4 angles 6"×4"×3/8", 2 plates 14"×3/8"	Web-plate 14"×3/8": 4 angles 6"×4"×7/16", 2 plates 14"×3/8"	Web-plate 14"×3/8": 4 angles 6"×4"×1/2", 2 plates 14"×3/8"	Web-plate 14"×3/8": 4 angles 6"×4"×1/2", 2 plates 14"×7/16"
11	1 085	1 130	392	422	452	474
12	1 085	1 130	392	422	452	474
13	1 085	1 130	392	422	452	474
14	1 085	1 130	392	422	452	474
15	1 085	1 130	392	422	452	474
16	1 085	1 130	387	415	444	470
17	1 085	1 130	375	403	431	456
18	1 082	1 130	363	390	417	442
19	1 054	1 101	352	377	404	428
20	1 026	1 072	340	365	390	415
21	998	1 043	328	352	377	401
22	970	1 014	317	340	363	387
23	942	985	305	327	350	373
24	914	956	293	314	336	359
25	886	927	281	302	323	345
26	858	898	270	289	309	331
27	830	869	258	276	296	317
28	802	840	246	264	282	303
29	774	811	235	251	269	289
30	746	782	223	239	255	275
31	718	753	211	227	243	261
32	690	725	205	220	236	251
33	662	696	200	214	229	244
34	634	667	194	208	222	237
35	606	638	188	201	216	230
Area, sq in	83.44	86.94	30.19	32.47	34.75	36.50
I_{1-1}, in⁴	3 486	3 721	1 261	1 351	1 436	1 539
r_{1-1}, in	6.46	6.54	6.46	6.45	6.43	6.49
I_{2-2}, in⁴	1071	1128	291	311	331	360
r_{2-2}, in	3.58	3.60	3.10	3.09	3.09	3.14
Weight, lb per lin ft	284.0	295.9	102.8	110.8	118.4	124.3

The safe load-values above the upper heavy line are for ratios of l/r not
those between heavy lines are for ratios up to 120 l/r; and those below the
heavy line are for ratios not over 200 l/r.

* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

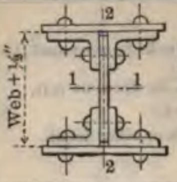
Weights do not include rivet-heads or other details

ve .	Web-plate 14"×3/8"		Web-plate 14"×1/2"		
	4 angles 6"×4"×1/2" 2 plates 14"×9/16"	4 angles 6"×4"×1/2" 2 plates 14"×5/8"	4 angles 6"×4"×1/2" 2 plates 14"×5/8"	4 angles 6"×4"×9/16" 2 plates 14"×5/8"	4 angles 6"×4"×5/8" 2 plates 14"×5/8"
	520	543	566	595	623
	520	543	566	595	623
	520	543	566	595	623
	520	543	566	595	623
	520	543	566	595	623
	520	543	566	595	623
	507	533	551	578	605
	493	517	535	561	587
	478	502	518	544	569
	463	487	502	527	551
	448	472	486	510	533
	433	456	470	493	515
	418	441	454	476	497
	403	426	437	459	479
	388	410	421	442	461
	374	395	405	424	443
	359	380	389	407	425
	344	364	373	390	407
	329	349	356	373	390
	314	334	340	356	372
	299	318	324	339	354
	284	303	308	322	336
	275	290	298	312	327
	267	282	290	304	318
	260	275	282	295	309
q in	40.00	41.75	43.50	45.74	47.94
	1 749	1 857	1 885	1 970	2 053
	6.61	6.67	6.58	6.56	6.54
	417	446	451	472	492
	3.23	3.27	3.22	3.21	3.20
ht, in ft..	136.2	142.2	148.1	155.7	163.3

afe load-values above the upper heavy line are for ratios of l/r not over 60; between the heavy lines are for ratios up to $120\,l/r$; and those below the eavy line are for ratios not over $200\,l/r$

Pocket Campanion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). **Safe Loads in Units of 1 000 Pounds for**
and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pound
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13)

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other detail

Effective length, ft	Web-plate 14"×⅝"					
	4 angles 6"×4"×⅝" 2 plates 14"×⅝"	4 angles 6"×4"×⅝" 2 plates 14"×¾"	4 angles 6"×4"×⅝" 2 plates 14"×⅞"	4 angles 6"×4"×⅝" 2 plates 14"×1"	4 angles 6"×4"×⅝" 2 plates 14"×1⅛"	4 angles 6"×4"×⅝" 2 plates 14"×1¼"
11	646	691	737	782	828	873
12	646	691	737	782	828	873
13	646	691	737	782	828	873
14	646	691	737	782	828	873
15	646	691	737	782	828	873
16	643	691	737	782	828	873
17	624	675	726	776	826	873
18	606	655	705	754	803	852
19	587	635	684	733	780	829
20	568	615	664	711	758	805
21	549	596	643	689	735	782
22	530	576	622	668	713	758
23	511	556	602	646	690	734
24	493	536	581	625	667	711
25	474	517	560	603	645	687
26	455	497	540	581	622	664
27	436	477	519	560	600	640
28	417	457	498	538	577	617
29	399	438	477	516	554	593
30	380	418	457	495	532	569
31	361	398	436	473	509	546
32	345	378	415	452	487	522
33	336	365	396	430	464	499
34	326	356	385	415	444	475
35	317	346	375	404	432	461
Area, sq in	49.69	53.19	56.69	60.19	63.69	67.19
I_{1-1}, in^4.....	2 081	2 302	2 529	2 764	3 006	3 255
r_{1-1}, in.....	6.47	6.58	6.68	6.78	6.87	6.96
I_{2-2}, in^4.....	499	556	613	671	728	785
r_{2-2}, in.....	3.17	3.23	3.29	3.34	3.38	3.42
Weight, lb per lin ft..	169.3	181.2	193.1	205.0	216.9	228.8

The safe load-values above the upper heavy line are for ratios of l/r not o
those between the heavy lines are for ratios up to 120 l/r; and those belo
lower heavy line are for ratios not over 200 l/r

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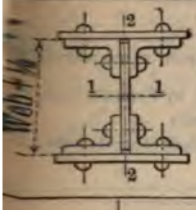
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsb

ble XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Web-plate 14"×5/8"					
	4 angles 6"×4"×5/8" 2 plates 14"×1½"	4 angles 6"×4"×5/8" 2 plates 14"×1⅝"	4 angles 6"×4"×5/8" 2 plates 14"×1¾"	4 angles 6"×4"×5/8" 2 plates 14"×1⅞"	4 angles 6"×4"×5/8" 2 plates 14"×2"	4 angles 6"×4"×5/8" 2 plates 16"×1⅞"
11	964	1 010	1 055	1 101	1 146	1 198
12	964	1 010	1 055	1 101	1 146	1 198
13	964	1 010	1 055	1 101	1 146	1 198
14	964	1 010	1 055	1 101	1 146	1 198
15	964	1 010	1 055	1 101	1 146	1 198
16	964	1 010	1 055	1 101	1 146	1 198
17	964	1 010	1 055	1 101	1 146	1 198
18	949	998	1 046	1 095	1 144	1 198
19	924	971	1 018	1 067	1 114	1 198
20	898	945	991	1 038	1 084	1 198
21	872	918	963	1 010	1 055	1 174
22	847	892	935	981	1 025	1 146
23	821	865	908	953	996	1 119
24	796	839	880	924	966	1 091
25	770	812	853	895	937	1 064
26	744	786	825	867	907	1 036
27	719	759	797	838	877	1 009
28	693	732	770	810	848	981
29	668	706	742	781	818	954
30	642	679	715	753	789	926
31	617	653	687	724	759	899
32	591	626	659	696	730	871
33	565	600	632	667	700	843
34	540	573	604	639	671	816
35	517	546	577	610	641	788
Area, sq in	74.19	77.69	81.19	84.69	88.19	92.19
$I_{1\text{-}1}$, in⁴.....	3 776	4 048	4 327	4 615	4 910	5 120
$r_{1\text{-}1}$, in.....	7.13	7.22	7.30	7.38	7.46	7.45
$I_{2\text{-}2}$, in⁴.....	899	956	1 014	1 071	1 128	1 493
$r_{2\text{-}2}$, in.....	3.48	3.51	3.53	3.56	3.58	4.02
Weight, lb per lin ft..	252.6	264.5	276.4	288.3	300.2	313.8

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

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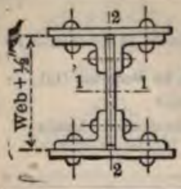
* *From Pocket Companion, 1915 Edition, Carnegie Steel Company,* **Pittsburgh, Pa.**

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for P and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pound
lengths of 60 radii or under
Reduced for lengths over 60 radii, by Formula (1
$S = 19\,000 - 100\,l/r$
Weights do not include rivet-heads or other detail

Effective length, ft	Web-plate 14″×⅝″				
	4 angles 6″×4″×⅝″ 2 plates 16″×2″	4 angles 6″×6″×⅝″ 2 plates 16″×2″	4 angles 6″×6″×⅝″ 2 plates 16″×2⅛″	4 angles 6″×6″×⅝″ 2 plates 16″×2¼″	4 angles 6″×6″×⅝″ 2 plates 16″×2⅜″
11	1 250	1 315	1 367	1 419	1 471
12	1 250	1 315	1 367	1 419	1 471
13	1 250	1 315	1 367	1 419	1 471
14	1 250	1 315	1 367	1 419	1 471
15	1 250	1 135	1 367	1 419	1 471
16	1 250	1 315	1 367	1 419	1 471
17	1 250	1 315	1 367	1 419	1 471
18	1 250	1 315	1 367	1 419	1 471
19	1 250	1 315	1 367	1 419	1 471
20	1 250	1 308	1 364	1 419	1 471
21	1 229	1 277	1 333	1 388	1 443
22	1 201	1 246	1 301	1 356	1 409
23	1 172	1 216	1 269	1 323	1 375
24	1 144	1 185	1 237	1 290	1 342
25	1 115	1 154	1 206	1 258	1 308
26	1 087	1 123	1 174	1 225	1 274
27	1 058	1 093	1 142	1 192	1 241
28	1 030	1 062	1 111	1 160	1 207
29	1 001	1 031	1 079	1 127	1 173
30	973	1 000	1 047	1 094	1 139
31	944	970	1 015	1 062	1 106
32	916	939	984	1 029	1 072
33	887	908	952	996	1 038
34	859	877	920	964	1 005
35	830	847	889	931	971
Area, sq in	96.19	101.19	105.19	109.19	113.19
I_{1-1}, in^4.....	5 457	5 484	5 830	6 187	6 552
r_{1-1}, in.....	7.53	7.36	7.44	7.53	7.61
I_{2-2}, in^4.....	1 579	1 581	1 666	1 752	1 837
r_{2-2}, in.....	4.05	3.95	3.98	4.01	4.03
Weight, lb per lin ft..	327.4	344.2	357.8	371.4	385.0

The safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

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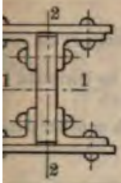
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

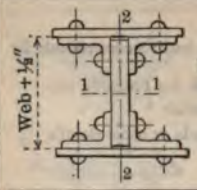
Weights do not include rivet-heads or other details

ve .	Two web-plates 14″×½″					
	4 angles 6″×6″×⅝″ 2 plates 16″×2½″	4 angles 8″×6″×⅝″ 2 plates 16″×2½″	4 angles 8″×6″×⅝″ 2 plates 18″×2⅜″	4 angles 8″×6″×⅝″ 2 plates 18″×2½″	4 angles 8″×6″×⅝″ 2 plates 18″×2⅝″	4 angles 8″×6″×⅝″ 2 plates 18″×2¾″
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 592	1 657	1 728	1 787	1 845	1 904
	1 590	1 657	1 728	1 787	1 845	1 904
	1 553	1 653	1 728	1 787	1 845	1 904
	1 516	1 616	1 728	1 787	1 845	1 904
	1 479	1 580	1 728	1 787	1 845	1 904
	1 443	1 543	1 695	1 756	1 818	1 879
	1 406	1 507	1 661	1 721	1 781	1 842
	1 369	1 470	1 626	1 685	1 744	1 804
	1 332	1 434	1 592	1 650	1 708	1 766
	1 295	1 397	1 557	1 614	1 671	1 729
	1 258	1 360	1 522	1 578	1 635	1 691
	1 222	1 324	1 488	1 543	1 598	1 653
	1 185	1 287	1 453	1 507	1 561	1 616
	1 148	1 251	1 419	1 471	1 525	1 578
	1 111	1 214	1 384	1 436	1 488	1 541
	1 074	1 177	1 349	1 400	1 451	1 503
	1 038	1 141	1 315	1 365	1 415	1 465
q in	122.44	127.44	132.94	137.44	141.94	146.44
.....	7 014	7 254	7 559	7 981	8 415	8 859
.....	7.57	7.54	7.54	7.62	7.70	7.78
.....	1 946	2 229	2 831	2 953	3 074	3 196
.....	3.99	4.18	4.61	4.63	4.65	4.67
t, n ft..	416.4	433.6	452.3	467.6	482.9	498.2

load-values above the heavy line are for ratios of l/r not over 60; those eavy line are for ratios not over 120 l/r

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds
and-Angle Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other

Effective length, ft	Two web-plates 14″×⅝″ 4 angles 8″×6″×⅝″ 2 plates 18″×2¾″	4 angles 8″×6″×⅝″ 2 plates 20″×2⅝″	4 angles 8″×6″×⅝″ 2 plates 20″×2¾″	4 angles 8″×6″×⅝″ 2 plates 20″×2⅞″	4 angles 8″×6″×⅝″ 2 plates 20″×3″
11	1 949	2 027	2 092	2 157	2 222
12	1 949	2 027	2 092	2 157	2 222
13	1 949	2 027	2 092	2 157	2 222
14	1 949	2 027	2 092	2 157	2 222
15	1 949	2 027	2 092	2 157	2 222
16	1 949	2 027	2 092	2 157	2 222
17	1 949	2 027	2 092	2 157	2 222
18	1 949	2 027	2 092	2 157	2 222
19	1 949	2 027	2 092	2 157	2 222
20	1 949	2 027	2 092	2 157	2 222
21	1 949	2 027	2 092	2 157	2 222
22	1 949	2 027	2 092	2 157	2 222
23	1 949	2 027	2 092	2 157	2 222
24	1 918	2 027	2 092	2 157	2 222
25	1 879	2 027	2 092	2 157	2 222
26	1 841	2 009	2 077	2 146	2 214
27	1 802	1 972	2 039	2 107	2 175
28	1 763	1 935	2 002	2 068	2 135
29	1 724	1 899	1 964	2 029	2 095
30	1 686	1 862	1 926	1 991	2 055
31	1 647	1 825	1 889	1 952	2 016
32	1 608	1 789	1 851	1 913	1 976
33	1 569	1 752	1 813	1 874	1 936
34	1 530	1 715	1 775	1 836	1 896
35	1 492	1 679	1 738	1 797	1 857
Area, sq in	149.94	155.94	160.94	165.94	170.94
I_{1-1}, in^4	8 916	9 248	9 741	10 248	10 767
r_{1-1}, in	7.71	7.70	7.78	7.86	7.94
I_{2-2}, in^4	3 222	4 049	4 216	4 383	4 549
r_{2-2}, in	4.64	5.10	5.12	5.14	5.16
Weight, lb per lin ft..	510.1	530.5	547.5	564.5	581.5

Safe load-values above the heavy line are for ratios of l/r not over
below the heavy line are for ratios not over 120 l/r

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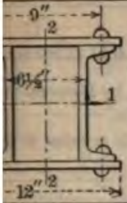
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

XXV.* Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

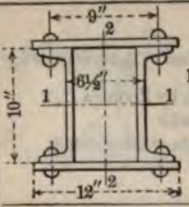
Weights do not include rivet-heads or other details

	Two 10-in channels latticed			Two 10-in channels, two 12-in plates					
ive h,	15-lb channels, single lattice	20-lb channels, single lattice	25-lb channels, single lattice	15-lb channels, $\frac{5}{16}$-in plates	15-lb channels, $\frac{3}{8}$-in plates	15-lb channels, $\frac{7}{16}$-in plates	15-lb channels, $\frac{1}{2}$-in plates	20-lb channels, $\frac{7}{16}$-in plates	20-lb channels, $\frac{1}{2}$-in plates
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	153	191	213	233	252	272	289	309
	116	152	186	213	233	252	271	286	305
	115	148	181	208	227	245	264	278	297
	112	144	176	203	221	239	257	271	289
	109	140	171	197	215	232	250	263	280
	106	136	165	192	209	226	243	256	272
	103	132	160	186	203	219	236	248	264
	100	128	155	181	197	213	229	240	256
	98	124	150	175	191	206	222	233	248
	95	120	145	170	185	200	215	225	240
	92	116	140	164	179	193	208	217	231
	89	112	134	159	173	187	201	210	223
	86	108	129	153	167	180	194	202	215
	83	104	124	148	161	174	187	195	207
	80	100	119	142	155	167	180	187	199
	77	96	114	137	149	161	173	179	191
	75	92	109	131	143	154	166	172	183
	72	88	103	126	137	148	159	164	174
	69	84	101	120	131	141	152	157	166
q in	8.92	11.76	14.70	16.42	17.92	19.42	20.92	22.26	23.76
.....	134	158	182	333	376	420	465	444	489
....	3.87	3.66	3.52	4.50	4.58	4.65	4.71	4.46	4.53
.....	123	148	171	213	231	249	267	274	292
....	3.72	3.55	3.41	3.60	3.59	3.58	3.58	3.51	3.50
t, n ft..	37.8	47.8	57.8	55.5	60.6	65.7	70.8	75.7	80.8

oad-values above the upper heavy line are for ratios of l/r not over 60; etween the heavy lines are for ratios up to 120 l/r; and those below the avy line are for ratios not over 200 l/r

Pocket *Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh,* Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for
Channel-Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

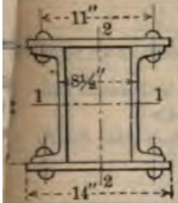
Weights do not include rivet-heads or other

Effective length, ft	Two 10-in channels, two 12-in plates						
	20-lb channels, 9/16-in plates	20-lb channels, 5/8-in plates	25-lb channels, 9/16-in plates	25-lb channels, 5/8-in plates	30-lb channels, 9/16-in plates	30-lb channels, 5/8-in plates	35-lb channels, 9/16-in plates
11	328	348	367	386	405	424	443
12	328	348	367	386	405	424	443
13	328	348	367	386	405	424	443
14	328	348	367	386	405	424	443
15	328	348	367	386	405	424	443
16	328	348	367	386	405	424	443
17	328	348	367	386	403	423	437
18	324	343	359	378	392	411	424
19	315	334	349	367	381	399	412
20	307	325	339	357	370	388	400
21	298	316	329	347	359	376	387
22	289	307	319	336	348	364	375
23	281	297	310	326	337	353	362
24	272	288	300	316	326	341	350
25	263	279	290	305	314	330	338
26	255	270	280	295	303	318	325
27	246	261	270	285	292	306	313
28	237	252	260	274	281	295	301
29	229	242	251	264	270	283	288
30	220	233	241	253	259	271	276
31	211	224	231	243	248	260	263
32	203	215	221	233	237	248	251
33	194	206	211	222	226	237	239
34	185	196	201	212	216	227	232
35	177	187	194	205	211	221	226
Area, sq in	25.26	26.76	28.20	29.70	31.14	32.64	34.08
I_{1-1}, in⁴	534	581	559	606	583	630	608
r_{1-1}, in	4.60	4.66	4.45	4.52	4.33	4.39	4.22
I_{2-2}, in⁴	310	328	333	351	354	372	372
r_{2-2}, in	3.50	3.50	3.44	3.44	3.37	3.37	3.30
Weight, lb per lin ft	85.9	91.0	95.9	101.0	105.9	111.0	115.9

Safe load-values above the upper heavy line are for ratios of l/r not over; those between the heavy lines are for ratios up to 120 l/r; and those below lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

le XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 10-in channels, latticed				Two 10-in channels, two 14-in plates			
	15-lb channels, single lattice	20-lb channels, single lattice	25-lb channels, single lattice	30-lb channels, single lattice	15-lb channels, ⅜-in plates	15-lb channels, 7/16-in plates	15-lb channels, ½-in plates	20-lb channels, 7/16-in plates
11	116	153	191	229	252	275	298	312
12	116	153	191	229	252	275	298	312
13	116	153	191	229	252	275	298	312
14	116	153	191	229	252	275	298	312
15	116	153	191	229	252	275	298	312
16	116	153	191	229	252	275	298	312
17	116	153	191	229	252	275	298	312
18	116	153	189	224	252	275	298	312
19	116	150	184	218	252	275	298	312
20	114	146	179	211	252	275	298	312
21	111	142	174	205	252	275	298	312
22	109	139	169	199	251	273	295	308
23	106	135	164	193	246	267	289	302
24	103	131	159	187	241	261	282	295
25	100	127	154	180	235	256	276	288
26	98	123	149	174	230	250	270	282
27	95	119	144	168	225	244	263	275
28	92	115	139	162	219	238	257	268
29	89	112	134	156	214	232	250	261
30	87	108	129	149	209	226	244	255
31	84	104	124	143	203	220	238	248
32	81	100	119	137	198	214	231	241
33	78	96	114	131	193	209	225	235
34	75	92	109	125	187	203	219	228
35	73	88	104	121	182	197	212	221
Area, sq in	8.92	11.76	14.70	17.64	19.42	21.17	22.92	24.01
I_{1-1}, in^4	134	158	182	207	416	468	520	491
r_{1-1}, in	3.87	3.66	3.52	3.42	4.63	4.70	4.76	4.52
I_{2-2}, in^4	197	241	284	323	369	398	426	442
r_{2-2}, in	4.70	4.53	4.39	4.28	4.36	4.33	4.31	4.29
Weight, lb per lin ft	39.3	49.4	59.4	69.4	65.7	71.7	77.6	81.7

Safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

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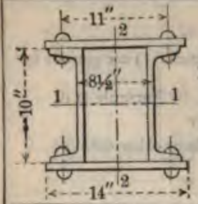
[*] From *Pocket Companion, 1915 Edition, Carnegie Steel Company*, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds
Channel-Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other

Effective length, ft	Two 10-in channels, two 14-in plates					
	20-lb channels, ½-in plates	20-lb channels, 9/16-in plates	20-lb channels, ⅝-in plates	25-lb channels, 9/16-in plates	25-lb channels, ⅝-in plates	25-lb channels, 11/16-in plates
11	335	358	380	396	419	441
12	335	358	380	396	419	441
13	335	358	380	396	419	441
14	335	358	380	396	419	441
15	335	358	380	396	419	441
16	335	358	380	396	419	441
17	335	358	380	396	419	441
18	335	358	380	396	419	441
19	335	358	380	396	419	441
20	335	358	380	396	419	441
21	335	358	380	396	419	441
22	330	352	374	388	410	432
23	323	344	365	379	401	422
24	316	337	357	371	392	412
25	308	329	349	362	382	403
26	301	321	341	353	373	393
27	294	313	332	345	364	383
28	287	306	324	336	355	373
29	279	298	316	327	346	364
30	272	290	308	319	336	354
31	265	282	299	310	327	344
32	258	275	291	301	318	335
33	251	267	283	293	309	325
34	243	259	274	284	300	315
35	236	251	266	275	291	306
Area, sq in	25.76	27.51	29.26	30.45	32.20	33.95
I_{1-1}, in^4	544	597	652	622	676	732
r_{1-1}, in	4.59	4.66	4.72	4.52	4.58	4.64
I_{2-2}, in^4	470	499	527	541	570	598
r_{2-2}, in	4.27	4.26	4.24	4.22	4.21	4.20
Weight, lb per lin ft	87.6	93.6	99.5	103.6	109.5	115.5

Safe load-values above the heavy line are for ratios of l/r not over 60; and below the heavy line are for ratios not over 120 l/r

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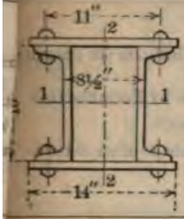
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

ble XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 10-in channels, two 14-in plates					
	30-lb channels, 11/16-in plates	30-lb channels, ¾-in plates	30-lb channels, 13/16-in plates	30-lb channels, ⅞-in plates	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates
11	480	502	525	548	571	593
12	480	502	525	548	571	593
13	480	502	525	548	571	593
14	480	502	525	548	571	593
15	480	502	525	548	571	593
16	480	502	525	548	571	593
17	480	502	525	548	571	593
18	480	502	525	548	571	593
19	480	502	525	548	571	593
20	480	502	525	548	571	593
21	477	500	522	544	567	589
22	467	488	510	532	554	575
23	456	477	499	520	541	562
24	446	466	487	508	529	549
25	435	455	475	495	516	536
26	424	444	464	483	503	522
27	414	432	452	471	490	509
28	403	421	440	459	478	496
29	392	410	429	446	465	483
30	382	399	417	434	452	469
31	371	388	405	422	440	456
32	360	377	394	410	427	443
33	350	365	382	398	414	430
34	339	354	370	385	401	416
35	328	343	359	373	389	403
Area, sq in	36.89	38.64	40.39	42.14	43.89	45.64
I_{1-1}, in^4.....	757	814	873	932	994	1 056
r_{1-1}, in.....	4.53	4.59	4.65	4.70	4.76	4.81
I_{2-2}, in^4.....	637	666	695	723	752	780
r_{2-2}, in.....	4.16	4.15	4.15	4.14	4.14	4.13
Weight, b per lin ft..	125.5	131.4	137.4	143.3	149.3	155.2

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

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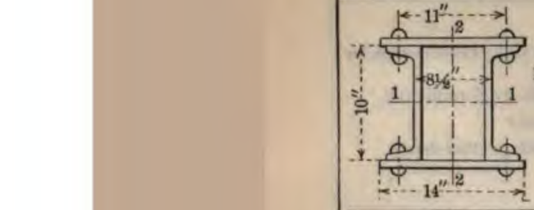
* From Pocket *Companion, 1915 Edition, Carnegie Steel Company*, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for
Channel-Columns



Allowable fiber-stress per square inch: 13 000 po
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (1

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other det

Effective length, ft	Two 10-in channels, two 14-in plates				
	35-lb channels, $\frac{15}{16}$-in plates	35-lb channels, 1-in plates	35-lb channels, $1\frac{1}{16}$-in plates	35-lb channels, $1\frac{1}{8}$-in plates	35-lb channels, $1\frac{3}{16}$-in plates
11	609	632	654	677	700
12	609	632	654	677	700
13	609	632	654	677	700
14	609	632	654	677	700
15	609	632	654	677	700
16	609	632	654	677	700
17	609	632	654	677	700
18	609	632	654	677	700
19	609	632	654	677	700
20	609	632	654	677	700
21	602	624	647	669	691
22	588	610	632	654	675
23	575	596	617	639	660
24	561	582	603	624	644
25	547	568	588	608	628
26	533	553	573	593	612
27	520	539	559	578	596
28	506	525	544	563	581
29	492	511	529	547	565
30	479	496	514	532	549
31	465	482	500	517	533
32	451	468	485	502	517
33	437	454	470	487	502
34	424	440	455	471	486
35	410	425	441	456	470
Area, sq in	46.83	48.58	50.33	52.08	53.83
I_{1-1}, in^4.....	1 018	1 080	1 144	1 209	1 275
r_{1-1}, in.....	4.66	4.72	4.77	4.82	4.87
I_{2-2}, in^4.....	788	816	845	874	902
r_{2-2}, in.....	4.10	4.10	4.10	4.10	4.09
Weight, lb per lin ft..	159.3	165.2	171.2	177.1	183.1

Safe load-values above the heavy line are for ratios of l/r not over 60; tho
low the heavy line are for ratios not over 120 l/r

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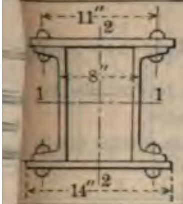
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburg

ble XXVI.* **Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns**



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

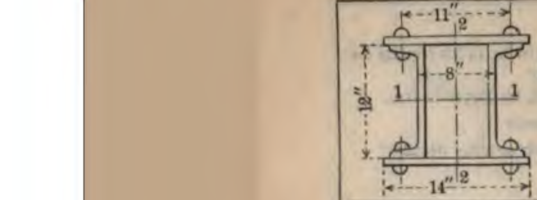
Weights do not include rivet-heads or other details

Effective length, ft	Two 12-in channels, latticed				Two 12-in channels, two 14-in plates		
	20½-lb channels, single lattice	25-lb channels, single lattice	30-lb channels, single lattice	35-lb channels, single lattice	20½-lb channels, ⅜-in plates	20½-lb channels, $\frac{7}{16}$-in plates	20½-lb channels, ½-in plates
11	157	191	229	268	293	316	339
12	157	191	229	268	293	316	339
13	157	191	229	268	293	316	339
14	157	191	229	268	293	316	339
15	157	191	229	268	293	316	339
16	157	191	229	268	293	316	339
17	157	191	229	268	293	316	339
18	157	191	229	268	293	316	339
19	157	191	229	268	293	316	339
20	157	191	229	268	293	316	339
21	157	191	229	265	293	316	339
22	157	190	225	259	290	312	334
23	155	186	220	253	283	305	326
24	152	182	215	248	277	298	319
25	149	178	210	242	271	291	312
26	146	174	205	236	265	284	304
27	142	170	200	230	258	277	297
28	139	166	195	224	252	271	290
29	136	162	190	218	246	264	282
30	133	158	185	212	239	257	275
31	129	154	180	206	233	250	268
32	126	150	175	200	227	243	260
33	123	146	170	194	220	236	253
34	120	142	165	188	214	230	246
35	117	138	160	182	208	223	238
rea, sq in	12.06	14.70	17.64	20.58	22.56	24.31	26.06
$_{-1}$, in^4.....	256	288	323	359	658	730	803
$_{-1}$, in.....	4.61	4.43	4.28	4.17	5.40	5.48	5.55
$_{-2}$, in^4.....	244	279	316	351	415	444	473
$_{-2}$, in.....	4.50	4.36	4.23	4.13	4.29	4.27	4.26
Weight, per lin ft..	50.4	59.4	69.4	79.4	76.7	82.7	88.6

Safe load-values above the heavy line are for ratios of l/r not over 60; those elow the heavy line are for ratios not over 120 l/r

From Pocket Companion, *1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

Table XXVI* (Continued). **Safe Loads in Units of 1 000 Pounds for**
Channel-Columns



Allowable fiber-stress per square inch: 13 000 pou lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (1

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other deta

Effective length, ft	Two 12-in channels, two 14-in plates					
	20½-lb channels, 9/16-in plates	20½-lb channels, ⅝-in plates	25-lb channels, 9/16-in plates	25-lb channels, ⅝-in plates	25-lb channels, 11/16-in plates	25-lb channels, ¾-in plates
11	362	384	396	419	441	464
12	362	384	396	419	441	464
13	362	384	396	419	441	464
14	362	384	396	419	441	464
15	362	384	396	419	441	464
16	362	384	396	419	441	464
17	362	384	396	419	441	464
18	362	384	396	419	441	464
19	362	384	396	419	441	464
20	362	384	396	419	441	464
21	362	384	396	418	440	463
22	355	377	387	409	431	453
23	347	369	378	400	421	443
24	339	360	370	390	411	432
25	332	352	361	381	401	422
26	324	344	352	372	392	412
27	316	335	344	363	382	402
28	308	327	335	354	372	391
29	300	318	326	344	362	381
30	292	310	318	335	353	371
31	284	302	309	326	343	361
32	277	293	300	317	333	350
33	269	285	291	307	323	340
34	261	277	283	298	314	330
35	253	268	274	289	304	320
Area, sq in	27.81	29.56	30.45	32.20	33.95	35.70
I_{1-1}, in^4.....	878	954	910	986	1 063	1 142
r_{1-1}, in.....	5.62	5.68	5.47	5.53	5.60	5.66
I_{2-2}, in^4.....	501	530	537	565	594	622
r_{2-2}, in.....	4.24	4.23	4.20	4.19	4.18	4.18
Weight, lb per lin ft..	94.6	100.5	103.6	109.5	115.5	121.4

Safe load-values above the heavy line are for ratios of l/r not over 60 below the heavy line are for ratios not over 120 l/r

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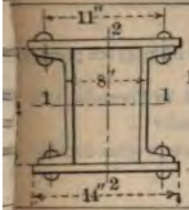
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

	Two 12-in channels, two 14-in plates							
ffective length, ft	30-lb channels, ¾-in plates	30-lb channels, 13/16-in plates	30-lb channels, ⅞-in plates	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates	35-lb channels, 15/16-in plates	35-lb channsel, 1-in plates	35-lb channels, 1 1/16-in plates
11	502	525	548	571	593	609	632	654
12	502	525	548	571	593	609	632	654
13	502	525	548	571	593	609	632	654
14	502	525	548	571	593	609	632	654
15	502	525	548	571	593	609	632	654
16	502	525	548	571	593	609	632	654
17	502	525	548	571	593	609	632	654
18	502	525	548	571	593	609	632	654
19	502	525	548	571	593	609	632	654
20	502	525	548	571	593	609	632	654
21	498	521	543	565	588	601	623	645
22	487	509	531	553	575	587	609	631
23	476	497	518	540	561	573	.594	616
24	465	486	506	527	548	559	580	601
25	453	474	494	514	535	545	566	586
26	442	462	482	502	522	532	552	571
27	431	451	469	489	508	518	537	557
28	420	439	457	476	495	504	523	542
29	409	427	445	463	482	490	509	527
30	397	415	432	450	468	477	494	512
31	386	404	420	438	455	463	480	497
32	375	392	408	425	442	449	466	483
33	364	380	396	412	428	435	452	468
34	352	368	383	399	415	421	437	453
35	341	357	371	386	402	408	423	438
rea, sq in	38.64	40.39	42.14	43.89	45.64	46.83	48.58	50.33
-1, in^4	1 174	1 258	1 340	1 424	1 509	1 459	1 544	1 630
-1, in	5.52	5.58	5.64	5.70	5.75	5.58	5.64	5.69
-2, in^4	659	688	717	745	774	779	808	837
-2, in	4.13	4.13	4.12	4.12	4.12	4.08	4.08	4.08
Weight, per lin ft..	131.4	137.4	143.3	149.3	155.2	159.3	165.2	171.2

Safe load-values above the heavy line are for ratios of l/r not over 60; those elow the heavy line are for ratios not over 120 l/r

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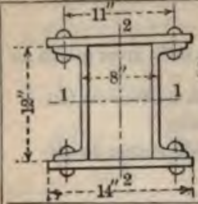
[*] From Pocket Companion, *1915 Edition, Carnegie Steel Company, Pittsburgh,* **Pa.**

Table XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for
Channel-Columns



Allowable fiber-stress per square inch: 13 000 lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other

Effective length, ft	Two 12-in channels, two 14-in plates					
	35-lb channels, 1⅛-in plates	35-lb channels, 1 3/16-in plates	35-lb channels, 1¼-in plates	35-lb channels, 1 5/16-in plates	35-lb channels, 1⅜-in plates	35-lb channels, 1 7/16-in plates
11	677	700	723	745	768	791
12	677	700	723	745	768	791
13	677	700	723	745	768	791
14	677	700	723	745	768	791
15	677	700	723	745	768	791
16	677	700	723	745	768	791
17	677	700	723	745	768	791
18	677	700	723	745	768	791
19	677	700	723	745	768	791
20	677	700	723	745	768	791
21	668	689	712	734	757	779
22	653	674	695	717	739	761
23	637	658	679	700	722	743
24	622	642	663	684	704	725
25	607	626	646	667	687	707
26	591	610	630	650	670	689
27	576	594	614	633	652	672
28	561	578	597	616	635	654
29	545	563	581	599	617	636
30	530	547	564	582	600	618
31	515	531	548	565	583	600
32	499	515	532	548	565	582
33	484	499	515	531	548	564
34	469	483	499	515	530	546
35	453	467	482	498	513	528
Area, sq in	52.08	53.83	55.58	57.33	59.08	60.83
I_{1-1}, in^4	1 719	1 808	1 899	1 992	2 087	2 183
r_{1-1}, in	5.74	5.80	5.85	5.89	5.94	5.99
I_{2-2}, in^4	865	894	922	951	980	1 008
r_{2-2}, in	4.08	4.07	4.07	4.07	4.07	4.07
Weight, lb per lin ft	177.1	183.1	189.0	195.0	200.9	206.9

Safe load-values above the heavy line are for ratios of l/r not over 60; below the heavy line are for ratios not over 120 l/r

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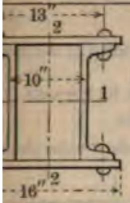
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh,

XVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

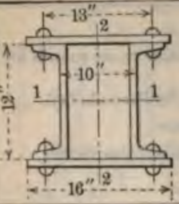
Weights do not include rivet-heads or other details

ive h,	Two 12-in channels, two 16-in plates									
	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates	30-lb channels, 1 1/16-in plates	30-lb channels, 1 1/8-in plates	30-lb channels, 1 3/16 in plates	30-lb channels, 1 1/4-in plates	35-lb channels, 1 3/16-in plates	35-lb channels, 1 1/4-in plates	35-lb channels, 1 5/16-in plates	35-lb channels, 1 3/8-in plates
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	788	814	840
	619	645	671	697	723	749	762	787	813	838
	610	635	660	686	711	736	747	772	797	822
	599	623	648	673	697	721	732	756	781	805
	587	611	635	659	683	707	718	741	766	789
	575	599	622	646	669	693	703	726	750	773
	563	586	609	633	655	678	688	711	734	757
	552	574	596	619	642	664	674	696	719	741
	540	562	583	606	628	649	659	681	703	724
	528	549	571	593	614	635	644	665	687	708
	516	537	558	579	600	621	630	650	672	692
	504	525	545	566	586	606	615	635	656	676
	493	512	532	553	572	592	600	620	640	660
q in	47.64	49.64	51.64	53.64	55.64	57.64	58.58	60.58	62.58	64.58
.....	1 581	1 678	1 777	1 878	1 980	2 084	2 015	2 119	2 225	2 333
.....	5.76	5.81	5.87	5.92	5.97	6.01	5.87	5.91	5.96	6.01
.....	1 121	1 164	1 206	1 249	1 292	1 334	1 349	1 392	1 434	1 477
.....	4.85	4.84	4.83	4.83	4.82	4.81	4.80	4.79	4.79	4.78
it, n ft..	162.0	168.8	175.6	182.4	189.2	196.0	199.2	206.0	212.8	219.6

oad-values above the heavy line are for ratios of l/r not over 60; those be- heavy line are for ratios not over 120 l/r

a Pocket *Companion, 1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

Table XXVI * (Continued). **Safe Loads in Units of 1 000 Pounds**
Channel-Columns



Allowable fiber-stress per square inch: 13 000 lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formu

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other

Effective length, ft	Two 12-in channels, two 16-in plates							
	35-lb channels, $1\frac{7}{16}$-in plates	35-lb channels, $1\frac{1}{2}$-in plates	35-lb channels, $1\frac{9}{16}$-in plates	35-lb channels, $1\frac{5}{8}$-in plates	35-lb channels, $1\frac{11}{16}$-in plates	35-lb channels, $1\frac{3}{4}$-in plates	35-lb channels, $1\frac{13}{16}$-in plates	35-lb channels, $1\frac{7}{8}$-in plates
11	866	892	918	944	970	996	1 022	1 048
12	866	892	918	944	970	996	1 022	1 048
13	866	892	918	944	970	996	1 022	1 048
14	866	892	918	944	970	996	1 022	1 048
15	866	892	918	944	970	996	1 022	1 048
16	866	892	918	944	970	996	1 022	1 048
17	866	892	918	944	970	996	1 022	1 048
18	866	892	918	944	970	996	1 022	1 048
19	866	892	918	944	970	996	1 022	1 048
20	866	892	918	944	970	996	1 022	1 048
21	866	892	918	944	970	996	1 022	1 048
22	866	892	918	944	970	996	1 022	1 048
23	866	892	918	944	970	996	1 022	1 048
24	864	889	915	940	966	992	1 017	1 042
25	847	872	897	922	947	972	997	1 022
26	830	854	879	903	928	953	977	1 002
27	814	837	862	885	909	934	957	981
28	797	820	844	867	891	914	937	961
29	780	803	826	848	872	895	917	941
30	764	785	808	830	853	876	897	920
31	747	768	791	812	834	857	878	900
32	730	751	773	794	815	837	858	880
33	713	734	755	775	797	818	838	859
34	697	716	737	757	778	799	818	839
35	680	699	720	739	759	779	798	819
Area, sq in	66.58	68.58	70.58	72.58	74.58	76.58	78.58	80.58
I_{1-1}, in^4.....	2 443	2 555	2 668	2 783	2 901	3 020	3 141	3 264
r_{1-1}, in.....	6.06	6.10	6.15	6.19	6.24	6.28	6.32	6.36
I_{2-2}, in^4.....	1 520	1 562	1 605	1 648	1 690	1 733	1 776	1 818
r_{2-2}, in.....	4.78	4.77	4.77	4.76	4.76	4.76	4.75	4.75
Weight, lb per lin ft..	226.4	233.2	240.0	246.8	253.6	260.4	267.2	274.0

Safe load-values above the heavy line are for ratios of l/r not over below the heavy line are for ratios not over 120 l/r

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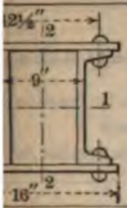
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitts

XXVII.* Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details

ive h,	Two 15-in channels, latticed				Two 15-in channels, two 16-in plates		
	33-lb channels, single lattice	35-lb channels, single lattice	40-lb channels, single lattice	45-lb channels, single lattice	33-lb channels, ⅜-in plates	33-lb channels, 7/16-in plates	33-lb channels, ½-in plates
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	344	413	439	465
	257	268	306	343	413	439	465
	257	266	301	336	407	432	457
	252	261	295	329	400	424	448
	247	256	289	322	392	415	440
	243	251	284	316	384	407	431
	238	246	278	309	376	399	422
	233	241	272	302	368	390	413
	228	236	266	296	360	382	404
	224	231	260	289	352	373	395
	219	226	254	282	345	365	386
	214	221	249	276	337	357	377
	209	216	243	269	329	348	368
q in	19.80	20.58	23.52	26.48	31.80	33.80	35.80
.....	625	640	695	750	1 334	1 459	1 586
.....	5.62	5.58	5.43	5.32	6.48	6.57	6.66
.....	491	504	552	597	747	789	832
.....	4.98	4.95	4.84	4.75	4.85	4.83	4.82
it, n ft..	80.2	84.2	92.1	102.2	106.8	113.6	120.4

load-values above the heavy line are for ratios of l/r not over 60; those he heavy line are for ratios not over 120 l/r

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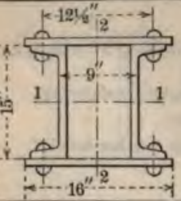
Pocket *Companion, 1915 Edition, Carnegie Steel Company*, Pittsburgh, Pa.

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds
Channel-Columns



Allowable fiber-stress per square inch: 15 000 lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

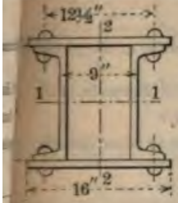
Weights do not include rivet-heads or other

Effective length, ft	Two 15-in channels, two 16-in plates					
	33-lb channels, 9/16-in plates	33-lb channels, 5/8-in plates	35-lb channels, 5/8-in plates	35-lb channels, 11/16-in plates	35-lb channels, 3/4-in plates	35-lb channels, 13/16-in plates
11	491	517	528	554	580	606
12	491	517	528	554	580	606
13	491	517	528	554	580	606
14	491	517	528	554	580	606
15	491	517	528	554	580	606
16	491	517	528	554	580	606
17	491	517	528	554	580	606
18	491	517	528	554	580	606
19	491	517	528	554	580	606
20	491	517	528	554	580	606
21	491	517	528	554	580	606
22	491	517	528	554	580	606
23	491	517	528	554	580	606
24	491	517	527	552	578	604
25	482	507	517	542	567	592
26	473	498	507	531	555	580
27	464	488	497	520	544	569
28	454	478	486	510	533	557
29	445	468	476	499	522	545
30	435	458	466	488	511	533
31	426	448	456	478	499	522
32	416	438	446	467	488	510
33	407	428	436	456	477	498
34	398	418	425	446	466	487
35	388	408	415	435	454	475
Area, sq in	37.80	39.80	40.58	42.58	44.58	46.58
I_{1-1}, in^4.....	1 715	1 847	1 861	1 994	2 129	2 267
r_{1-1}, in.....	6.74	6.81	6.77	6.84	6.91	6.98
I_{2-2}, in^4.....	875	917	930	973	1 016	1 058
r_{2-2}, in.....	4.81	4.80	4.79	4.78	4.77	4.77
Weight, lb per lin ft..	127.2	134.0	138.0	144.8	151.6	158.4

Safe load-values above the heavy line are for ratios of l/r not over 60; below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

le XXVII* (Continued). **Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns**



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 15-in channels, two 16-in plates						
	40-lb channels, 13/16-in plates	40-lb channels, 7/8-in plates	40-lb channels, 15/16-in plates	40-lb channels, 1-in plates	40-lb channels, 1 1/16-in plates	40-lb channels, 1 1/8-in plates	45-lb channels, 1 1/16-in plates
11	644	670	696	722	748	774	786
12	644	670	696	722	748	774	786
13	644	670	696	722	748	774	786
14	644	670	696	722	748	774	786
15	644	670	696	722	748	774	786
16	644	670	696	722	748	774	786
17	644	670	696	722	748	774	786
18	644	670	696	722	748	774	786
19	644	670	696	722	748	774	786
20	644	670	696	722	748	774	786
21	644	670	696	722	748	774	786
22	644	670	696	722	748	774	786
23	644	670	696	722	748	774	786
24	639	665	690	715	741	767	777
25	627	651	677	701	727	752	761
26	614	638	663	687	712	737	746
27	602	625	649	673	697	721	730
28	589	612	636	659	683	706	715
29	577	599	622	645	668	691	699
30	564	586	609	631	653	676	684
31	551	573	595	616	639	661	668
32	539	560	581	602	624	646	653
33	526	547	568	588	609	630	637
34	514	534	554	574	595	615	622
35	501	520	541	560	580	600	606
rea, sq in	49.52	51.52	53.52	55.52	57.52	59.52	60.48
$_{1-1}$, in^4	2 322	2 461	2 602	2 746	2 891	3 039	2 946
$_{1-1}$, in	6.85	6.91	6.97	7.03	7.09	7.15	6.98
$_{2-2}$, in^4	1 106	1 149	1 192	1 234	1 277	1 320	1 322
$_{2-2}$, in	4.73	4.72	4.72	4.71	4.71	4.71	4.68
Weight, per lin ft	168.4	175.2	182.0	188.8	195.6	202.4	205.6

Safe load-values above the heavy line are for ratios of l/r not over 60; those elow the heavy line are for ratios not over 120 l/r

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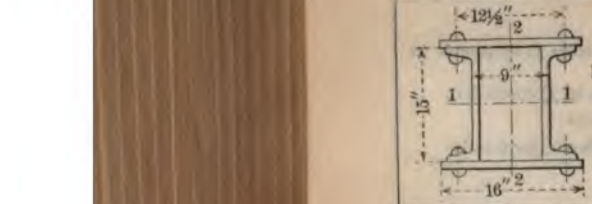
* From Pocket *Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds
Channel-Columns



Allowable fiber-stress per square inch: 13 000
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other det

Effective length, ft	Two 15-in channels, two 16-in plates					
	45-lb channels, 1⅛-in plates	45-lb channels, 1³⁄₁₆-in plates	45-lb channels, 1¼-in plates	45-lb channels, 1⁵⁄₁₆-in plates	45-lb channels, 1⅜-in plates	45-lb channels, 1⁷⁄₁₆-in plates
11	812	838	864	890	916	942
12	812	838	864	890	916	942
13	812	838	864	890	916	942
14	812	838	864	890	916	942
15	812	838	864	890	916	942
16	812	838	864	890	916	942
17	812	838	864	890	916	942
18	812	838	864	890	916	942
19	812	838	864	890	916	942
20	812	838	864	890	916	942
21	812	838	864	890	916	942
22	812	838	864	890	916	942
23	812	838	864	890	916	942
24	802	827	853	879	904	930
25	786	811	836	861	886	912
26	770	794	819	844	868	893
27	754	778	802	826	850	874
28	738	761	785	808	832	856
29	722	745	768	791	814	837
30	705	728	751	773	796	818
31	689	711	734	756	778	800
32	673	695	716	738	760	781
33	657	678	699	720	741	763
34	641	662	682	703	723	744
35	625	645	665	685	705	725
Area, sq in	62.48	64.48	66.48	68.48	70.48	72.48
I_{1-1}, in^4.....	3 094	3 244	3 396	3 550	3 707	3 865
r_{1-1}, in.....	7.04	7.09	7.15	7.20	7.25	7.30
I_{2-2}, in^4.....	1 365	1 408	1 450	1 493	1 536	1 578
r_{2-2}, in.....	4.67	4.67	4.67	4.67	4.67	4.67
Weight, lb per lin ft..	212.4	219.2	226.0	232.8	239.6	246.4

Safe load-values above the heavy line are for ratios of l/r not over 60; below the heavy line are for ratios not over 120 l/r

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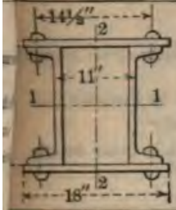
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

le XXVII * (Continued). Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

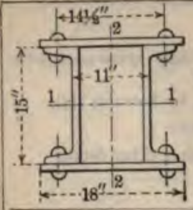
Weights do not include rivet-heads or other details

Effective length, ft	Two 15-in channels, two 18-in plates							
	33-lb channels, $\frac{3}{8}$-in plates	33-lb channels, $\frac{7}{16}$-in plates	33-lb channels, $\frac{1}{2}$-in plates	33-lb channels, $\frac{9}{16}$-in plates	33-lb channels, $\frac{5}{8}$-in plates	35-lb channels, $\frac{5}{8}$-in plates	35-lb channels, $\frac{11}{16}$-in plates	35-lb channels, $\frac{3}{4}$-in plates
11	433	462	491	521	550	560	589	619
12	433	462	491	521	550	560	589	619
13	433	462	491	521	550	560	589	619
14	433	462	491	521	550	560	589	619
15	433	462	491	521	550	560	589	619
16	433	462	491	521	550	560	589	619
17	433	462	491	521	550	560	589	619
18	433	462	491	521	550	560	589	619
19	433	462	491	521	550	560	589	619
20	433	462	491	521	550	560	589	619
21	433	462	491	521	550	560	589	619
22	433	462	491	521	550	560	589	619
23	433	462	491	521	550	560	589	619
24	433	462	491	521	550	560	589	619
25	433	462	491	521	550	560	589	619
26	433	462	491	521	550	560	589	619
27	433	462	491	521	550	560	589	619
28	433	462	491	520	549	558	586	615
29	428	456	484	512	539	549	577	605
30	421	449	476	503	530	540	567	594
31	414	441	468	494	521	530	557	584
32	407	433	459	486	512	521	547	574
33	400	426	451	477	503	512	537	563
34	393	418	443	469	494	502	527	553
35	386	411	435	460	485	493	518	543
Area, sq in	33.30	35.55	37.80	40.05	42.30	43.08	45.33	47.58
$_{1-1}$, in^4	1 423	1 564	1 707	1 852	1 999	2 014	2 164	2 316
$_{1-1}$, in	6.54	6.63	6.72	6.80	6.87	6.84	6.91	6.98
$_{2-2}$, in^4	1 069	1 130	1 190	1 251	1 312	1 332	1 393	1 453
$_{2-2}$, in	5.67	5.64	5.61	5.59	5.57	5.56	5.54	5.53
Weight, per lin ft...	111.9	119.6	127.2	134.9	142.5	146.5	154.2	161.8

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, *1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table XXVII* (Continued). **Safe Loads in Units of 1 000 Pounds for
Channel-Columns**



Allowable fiber-stress per square inch: 13 000 pounds
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 15-in channels, two 18-in plates 35-lb channels, 13/16-in plates	35-lb channels, 7/8-in plates	40-lb channels, 13/16-in plates	40-lb channels, 7/8-in plates	40-lb channels, 15/16-in plates	40-lb channels, 1-in plates	40-lb channels, 1 1/16-in plates
11	648	677	686	715	745	774	803
12	648	677	686	715	745	774	803
13	648	677	686	715	745	774	803
14	648	677	686	715	745	774	803
15	648	677	686	715	745	774	803
16	648	677	686	715	745	774	803
17	648	677	686	715	745	774	803
18	648	677	686	715	745	774	803
19	648	677	686	715	745	774	803
20	648	677	686	715	745	774	803
21	648	677	686	715	745	774	803
22	648	677	686	715	745	774	803
23	648	677	686	715	745	774	803
24	648	677	686	715	745	774	803
25	648	677	686	715	745	774	803
26	648	677	686	715	745	774	803
27	648	677	686	715	745	774	803
28	643	671	680	708	736	764	793
29	632	660	668	696	723	751	779
30	621	649	657	684	711	738	766
31	610	637	645	672	698	725	752
32	599	626	634	660	685	712	738
33	589	615	622	648	673	698	725
34	578	603	610	636	660	685	711
35	567	592	599	624	648	672	698
Area, sq in	49.83	52.08	52.77	55.02	57.27	59.52	61.77
I_{1-1}, in^4.....	2 470	2 627	2 525	2 682	2 841	3 002	3 166
r_{1-1}, in.....	7.04	7.10	6.92	6.98	7.04	7.10	7.16
I_{2-2}, in^4.....	1 514	1 575	1 589	1 649	1 710	1 771	1 832
r_{2-2}, in.....	5.51	5.50	5.49	5.48	5.46	5.45	5.45
Weight, lb per lin ft..	169.5	177.1	179.5	187.1	194.8	202.4	210.1

Safe load-values above the heavy line are for ratios of l/r not over 60;
below the heavy line are for ratios not over 120 l/r

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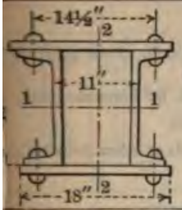
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

XXVII* **(Continued). Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns**



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 15-in channels, two 18-in plates						
	45-lb channels, 1 1/16-in plates	45-lb channels, 1 1/8-in plates	45-lb channels, 1 3/16-in plates	45-lb channels, 1 1/4-in plates	45-lb channels, 1 5/16-in plates	45-lb channels, 1 3/8-in plates	45-lb channels, 1 7/16-in plates
11	841	871	900	929	958	988	1 017
12	841	871	900	929	958	988	1 017
13	841	871	900	929	958	988	1 017
14	841	871	900	929	958	988	1 017
15	841	871	900	929	958	988	1 017
16	841	871	900	929	958	988	1 017
17	841	871	900	929	958	988	1 017
18	841	871	900	929	958	988	1 017
19	841	871	900	929	958	988	1 017
20	841	871	900	929	958	988	1 017
21	841	871	900	929	958	988	1 017
22	841	871	900	929	958	988	1 017
23	841	871	900	929	958	988	1 017
24	841	871	900	929	958	988	1 017
25	841	871	900	929	958	988	1 017
26	841	871	900	929	958	988	1 017
27	841	871	900	929	958	987	1 015
28	829	857	885	913	942	970	998
29	814	843	870	897	926	953	980
30	800	828	855	882	909	936	963
31	786	813	839	866	893	919	945
32	771	798	824	850	877	902	928
33	757	783	809	834	860	885	911
34	743	768	793	818	844	868	893
35	728	754	778	802	827	852	876
Area, sq in	64.73	66.98	69.23	71.48	73.73	75.98	78.23
I_{1-1}, in^4.....	3 221	3 387	3 556	3 727	3 900	4 076	4 255
r_{1-1}, in.....	7.05	7.11	7.17	7.22	7.27	7.32	7.37
I_{2-2}, in^4.....	1 903	1 964	2 025	2 086	2 146	2 207	2 268
r_{2-2}, in.....	5.42	5.42	5.41	5.40	5.40	5.39	5.38
Weight, b per lin ft..	220.1	227.7	235.4	243.0	250.0	258.3	266.0

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

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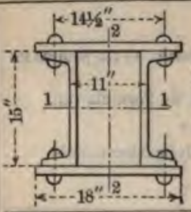
* From Pocket *Companion, 1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

Table XXVII[*] (Continued). Safe Loads in Units of 1 000 Pounds f
Channel-Columns



Allowable fiber-stress per square inch: 13 000 p
lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other de

Effective length, ft	Two 15-in channels, two 18-in plates					
	45-lb channels, 1½-in plates	45-lb channels, 1 9/16-in plates	45-lb channels, 1⅝-in plates	45-lb channels, 1 11/16-in plates	45-lb channels, 1¾-in plates	45-lb channels, 1⅞-in plates
11	1 046	1 075	1 105	1 134	1 163	1 222
12	1 046	1 075	1 105	1 134	1 163	1 222
13	1 046	1 075	1 105	1 134	1 163	1 222
14	1 046	1 075	1 105	1 134	1 163	1 222
15	1 046	1 075	1 105	1 134	1 163	1 222
16	1 046	1 075	1 105	1 134	1 163	1 222
17	1 046	1 075	1 105	1 134	1 163	1 222
18	1 046	1 075	1 105	1 134	1 163	1 222
19	1 046	1 075	1 105	1 134	1 163	1 222
20	1 046	1 075	1 105	1 134	1 163	1 222
21	1 046	1 075	1 105	1 134	1 163	1 222
22	1 046	1 075	1 105	1 134	1 163	1 222
23	1 046	1 075	1 105	1 134	1 163	1 222
24	1 046	1 075	1 105	1 134	1 163	1 222
25	1 046	1 075	1 105	1 134	1 163	1 222
26	1 046	1 075	1 105	1 134	1 163	1 222
27	1 044	1 073	1 102	1 131	1 159	1 216
28	1 026	1 054	1 083	1 112	1 139	1 195
29	1 009	1 036	1 064	1 092	1 119	1 174
30	991	1 017	1 045	1 073	1 099	1 153
31	973	999	1 026	1 053	1 079	1 132
32	955	980	1 007	1 034	1 059	1 111
33	937	962	988	1 014	1 039	1 090
34	919	943	969	995	1 019	1 069
35	901	925	950	975	999	1 048
Area, sq in	80.48	82.73	84.98	87.23	89.48	93.98
I_{1-1}, in^4	4 436	4 619	4 805	4 994	5 185	5 575
r_{1-1}, in	7.42	7.47	7.52	7.57	7.61	7.70
I_{2-2}, in^4	2 329	2 389	2 450	2 511	2 572	2 693
r_{2-2}, in	5.38	5.37	5.37	5.37	5.36	5.35
Weight, lb per lin ft	273.6	281.3	288.9	296.6	304.2	319.5

Safe load-values above the heavy line are for ratios of l/r not over 60
below the heavy line are for ratios not over 120 l/r

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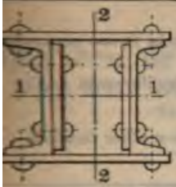
[*] From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress per square inch: 13 000 pounds for lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

ffective length, ft	Two 15-in channels			Two 15-in, 45-lb channels			
	35 lb		45 lb				
	2 flange-plates 18"×2" 2 web-plates 14"×⅜"	2 flange-plates 18"×2" 2 web-plates 14"×9/16"	2 flange-plates 18"×2" 2 web-plates 14"×9/16"	2 flange-plates 20"×1⅞" 2 web-plates 14"×⅝"	2 flange-plates 20"×2" 2 web-plates 14"×⅝"	2 flange-plates 20"×2⅛" 2 web-plates 14"×⅝"	2 flange-plates 20"×2¼" 2 web-plates 14"×⅝"
11	1 340	1 408	1 485	1 547	1 612	1 677	1 742
12	1 340	1 408	1 485	1 547	1 612	1 677	1 742
13	1 340	1 408	1 485	1 547	1 612	1 677	1 742
14	1 340	1 408	1 485	1 547	1 612	1 677	1 742
15	1 340	1 408	1 485	1 547	1 612	1 677	1 742
16	1 340	1 408	1 485	1 547	1 612	1 677	1 742
17	1 340	1 408	1 485	1 547	1 612	1 677	1 742
18	1 340	1 408	1 485	1 547	1 612	1 677	1 742
19	1 340	1 408	1 485	1 547	1 612	1 677	1 742
20	1 340	1 408	1 485	1 547	1 612	1 677	1 742
21	1 340	1 408	1 485	1 547	1 612	1 677	1 742
22	1 340	1 408	1 485	1 547	1 612	1 677	1 742
23	1 340	1 408	1 485	1 547	1 612	1 677	1 742
24	1 340	1 408	1 485	1 547	1 612	1 677	1 742
25	1 340	1 408	1 485	1 547	1 612	1 677	1 742
26	1 340	1 408	1 485	1 547	1 612	1 677	1 742
27	1 331	1 394	1 465	1 547	1 612	1 677	1 742
28	1 307	1 369	1 439	1 547	1 612	1 677	1 742
29	1 284	1 344	1 413	1 547	1 612	1 677	1 742
30	1 261	1 320	1 387	1 543	1 607	1 670	1 735
31	1 238	1 295	1 361	1 519	1 582	1 644	1 708
32	1 214	1 270	1 335	1 495	1 557	1 618	1 681
33	1 191	1 246	1 309	1 471	1 532	1 592	1 654
34	1 168	1 221	1 283	1 447	1 507	1 566	1 627
35	1 145	1 197	1 257	1 424	1 482	1 540	1 600
Area, sq in	103.08	108.33	114.23	118.98	123.98	128.98	133.98
I_{1-1}, in^4	6 037	6 123	6 233	6 397	6 843	7 300	7 769
r_{1-1}, in	7.65	7.52	7.39	7.33	7.43	7.52	7.61
I_{2-2}, in^4	2 919	3 021	3 148	4 240	4 407	4 573	4 740
r_{2-2}, in	5.32	5.28	5.25	5.97	5.96	5.95	5.95
Weight, lb per lin ft	350.5	368.4	388.4	404.5	421.5	438.5	455.5

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

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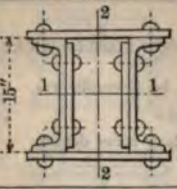
* From *Pocket Companion, 1915 Edition, Carnegie Steel Company,* **Pittsburgh, Pa.**

Table XXVII* (Concluded). Safe Loads in Units of 1 000 Pounds for **Channel-Columns**



Allowable fiber-stress per square inch: 13 000 pounds lengths of 60 radii or under

Reduced for lengths over 60 radii, by Formula (1

$$S = 19\,000 - 100\,l/r$$

Weights do not include rivet-heads or other details

Effective length, ft	Two 15-in, 45-lb channels				
	2 flange-plates 20"×2⅜" 2 web-plates 14"×⅝"	2 flange-plates 20"×2½" 2 web-plates 14"×⅝"	2 flange-plates 20"×2⅝" 2 web-plates 14"×⅝"	2 flange-plates 20"×2¾" 2 web-plates 14"×⅝"	2 flange-plates 20"×2⅞" 2 web-plates 14"×⅝"
11	1 807	1 872	1 937	2 002	2 067
12	1 807	1 872	1 937	2 002	2 067
13	1 807	1 872	1 937	2 002	2 067
14	1 807	1 872	1 937	2 002	2 067
15	1 807	1 872	1 937	2 002	2 067
16	1 807	1 872	1 937	2 002	2 067
17	1 807	1 872	1 937	2 002	2 067
18	1 807	1 872	1 937	2 002	2 067
19	1 807	1 872	1 937	2 002	2 067
20	1 807	1 872	1 937	2 002	2 067
21	1 807	1 872	1 937	2 002	2 067
22	1 807	1 872	1 937	2 002	2 067
23	1 807	1 872	1 937	2 002	2 067
24	1 807	1 872	1 937	2 002	2 067
25	1 807	1 872	1 937	2 002	2 067
26	1 807	1 872	1 937	2 002	2 067
27	1 807	1 872	1 937	2 002	2 067
28	1 807	1 872	1 937	2 002	2 067
29	1 807	1 872	1 937	2 002	2 067
30	1 798	1 863	1 926	1 991	2 054
31	1 770	1 834	1 896	1 960	2 022
32	1 742	1 805	1 866	1 929	1 989
33	1 714	1 776	1 836	1 897	1 957
34	1 686	1 747	1 806	1 866	1 925
35	1 658	1 718	1 775	1 835	1 893
Area, sq in	138.98	143.98	148.98	153.98	158.98
I_{1-1}, in^4	8 251	8 744	9 251	9 770	10 301
r_{1-1}, in	7.70	7.79	7.88	7.97	8.05
I_{2-2}, in^4	4 907	5 073	5 240	5 407	5 573
r_{2-2}, in	5.94	5.94	5.93	5.93	5.92
Weight, lb per lin ft	472.5	489.5	506.5	523.5	540.5

Safe load-values above the heavy line are for ratios of l/r not over 60; below the heavy line are for ratios not over 120 l/r

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburg

CHAPTER XV

GTH OF BEAMS AND BEAM GIRDERS. FRAMING AND CONNECTING STEEL BEAMS

By
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1. General Principles of the Flexure of Beams

ions. A structural member placed in a generally horizontal position
or more supports or projecting from some other construction is called
A GIRDER is a beam carrying smaller or secondary beams. A CANTI-
AM is a beam supported at the middle, or having one end fixed, as in a
the other end free; or it is the part of a beam which overhangs, or
beyond a support. A SIMPLE BEAM is one which rests upon two sup-
at each end. A CONTINUOUS BEAM rests upon more than two supports.
nce between the supports of a simple beam, or, when so specially desig-
m center to center of the bearings, is the SPAN. It is usually designated
he loads on beams are either UNIFORMLY DISTRIBUTED or CONCEN-
A uniformly distributed or UNIFORM load includes the weight of the
elf and any load spread evenly over it, such as the weight of a wall.
loads are estimated by their intensity per unit of length of the beam,
s per linear foot. A uniform load per linear foot is represented by w,
otal uniform load by wl or W. A concentrated load is a single applied
uch as a column and its load, or the load from another beam, and is
d by P.

es and Deformations. A load on a simple beam causes the fibers to
eflect, and eventually to break across, or in other words, a load induces
SE or FLEXURAL STRESSES in the fibers. Since it is impossible to bend
a simple beam without causing a shortening of the fibers on the upper
e side and an elongation of the fibers on the lower or convex side, a
beam causes COMPRESSION in the upper fibers and TENSION in the
rs, while between the two there is a neutral layer or surface of fibers
unchanged in length and which is called the NEUTRAL SURFACE of the
n a cantilever beam the reverse is the case, the upper fibers being
and the lower ones in compression.

Determined by Experiment. From experiments it has been found
amount of elongation or shortening of any fiber is directly propor-
its distance from the neutral surface of a beam; hence, if the ELASTIC
ot exceeded, the stresses, also, are proportional to their distances from
ral surface. The trace of the neutral surface on a cross-section of a
called the NEUTRAL AXIS of the cross-section. Within the elastic
material the neutral surface passes through the CENTERS OF GRAVITY
ss-sections of a beam for all materials.

g Moments and Resisting Moments.* To determine the strength
am to resist the effects of any load or series of loads, two things must

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* *See, also, Chapter IX, pages 324 and 325.*

the SECTION-MODULUS or SECTION-FACTOR, denoted by I/c, mult
unit stress on the outermost fiber of the material, denoted by S,
the bending moment.

Hence $$M = SI/c$$

This is known as the FLEXURE-FORMULA and it is the fundamenta
designing beams. Formulas for finding the section-moduli of co
are given in Chapter X, and the values of I/c or the section-m
standard rolled shapes, are given in the tables in the same chapter

The Coefficient of Strength,* sometimes given in tables of s
the maximum distributed load that a beam of one foot span w
without producing a fiber-stress exceeding the safe limit, genera
per sq in. As the strength of a beam varies inversely as its span,
for any span may be obtained by dividing this coefficient by the sp

Factors of Safety. In order that a beam shall just be able to
and not break, that condition of equilibrium must exist, in whi
mum bending moment in the beam is equal to the section-modul
by the ultimate strength of the material. In order that a b
abundantly SAFE to carry a given load, the product of the sectior
the ultimate strength of the material must be several times gre
maximum bending moment; and the ratio which this product
maximum bending moment, or which the BREAKING-LOAD bears
LOAD, is known as the FACTOR OF SAFETY, that is,

$$\text{Factor of safety} = \frac{\text{ultimate strength}}{\text{working stress}}$$

Ultimate Strengths and Safe Fiber-Stresses. By the STRE
MATERIAL is meant a certain constant quantity which is determin
ment, and which is known as the ULTIMATE BREAKING STRE
value is of course different for each material. Table I gives the v
constant divided by the factor of safety, or in other words, the WOR
for most of the materials used in building-construction. The se
multiplied by these values will give the SAFE RESISTING MOMENTS fo
The values of S in Table I for steel are about one-fourth those of

ue of 16 000 lb per sq in for *S*, but these full loads should be used
on, and reduced when necessary to satisfy any unusual conditions.
steel girders 14 000 lb per sq in is the value usually given to *S*.

able I. Safe Unit Fiber-Stresses, S, for Flexure of Beams*

to be noted that these are average values, especially those for wood

Materials	Values of *S*, lb per sq in	Materials	Values of *S*, lb per sq in
tension-side.....	3 000	Redwood, California......	750
compression-side	16 000	Short-leaf yellow pine......	1 000
iron (rolled		Spruce....................	700
....	12 000	White oak.................	1 200
ed beams)......	16 000	White pine................	700
ted girders, net ction)..........	14 000	Bluestone flagging (North River)..................	305
ns, rivets and		Brick (common)..........	50
....	20 000	Brickwork (in cement)....	30
....	700	Granite (average).........	180
....	800	Limestone (average).......	145
....	800	Marble (average)..........	125
r...............	800	Sandstone (average).......	110
....	900	Slate (average)...........	400
....	600	Concrete (Portland) 1 : 2 : 4	30
....	1 200	Concrete (Portland) 1 : 2 : 5	20
yellow pine......	1 200	Concrete (natural) 1:2:4...	16
ne..............	800	Concrete (natural) 1:2:5...	10

mparison of values given in different building laws see Table XVII, page 648, I. Compare, also, with Table XVI, page 647, Chapter XVI. For ultimate oods, see Tables XVIII and XIX, pages 650 and 651, Chapter XVI. For r unit beams, see Tables II and III, page 628, Chapter XVI.

Jnsymmetrically Loaded or of Irregular Cross-Section. There loadings and cross-sections of beams that occur most frequently construction, and for which tables have been worked out that give ds directly; but for a beam unsymmetrically loaded, or for a beam cross-section, it is impossible to compute tables for strength, as in the values must be computed by determining either the section-/*c*, required to resist the maximum bending moment, or the maxi-ing moment that may be allowed for a given value of the section-

Formulas for the Flexure of Beams.* The general formula for n a state of flexure under any system of loading is

um bending moment in INCH-POUNDS = section-modulus × *S* (2)

$$M_{max} = SI/c \qquad (2)'$$

$$\text{Section-modulus} = \frac{\text{maximum bending moment in in-lb}}{S} \qquad (3)$$

$$I/c = M_{max}/S \qquad (3)'$$

---

* *See, also, Chapters IX, X and XVI.*

If the bending moment is computed in FOOT-POUNDS, these form

$$\text{Maximum bending moment} = \frac{\text{section-modulus} \times S}{12}$$

or

$$M_{max} = SI/12\,c$$

and

$$\text{Section-modulus} = \frac{12 \times \text{maximum bending moment}}{S}$$

or

$$I/c = 12\,M_{max}/S$$

By substituting for the bending moments their values in terms o
and the spans, the following formulas which apply to beams of any c
are readily deduced.

2. Formulas for Safe Loads for Beams for Different Con of Loading and Support

I/c = the section-modulus;
S = the safe unit fiber-stress in pounds per square inch;
W = the total uniform load in pounds;
P = the concentrated load in pounds;
l = the span in feet.

Values of I/c for the various shapes and sizes of structural-steel
given in the tables of Chapter X.

Case I

Beam Fixed at One End and Loaded with a Concentrated Load P, Ne End (Fig. 1).

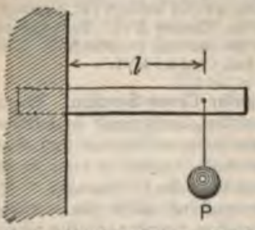


Fig. 1. Cantilever Beam. Load near Free End

From Formula (4)′,

$$M_{max} = SI/12\,c$$

From Case I, Chapter IX,

$$M_{max} = Pl$$

Hence

$$Pl = SI/12\,c$$

and the safe load in pounds is

$$P = SI/12\,d$$

and the section-modulus is

$$I/c = 12\,Pl/S$$

Example 1. A steel T bar is fixed at one end in a brick wall, and
the other end with 600 lb, the distance l being 4 ft. What is the size
required to support the load with safety? (In all examples the wei
beams are neglected, unless particularly mentioned.)

Solution. Allowing 16 000 lb per sq in for the value of S, Formula

$$I/c = (12 \times 600 \times 4)/16\,000 = 1.8$$

The next step is to ascertain what T bar has a section-modulus eq
In Table XIV, page 369, the nearest section-modulus to this is 1.9,
ing to a 3 by 4 by ½-in T bar.

For an **I** beam, by Table IV, page 355, $I/c = 1.8$, the same as for
and calls for a 3-in 6.5-lb **I** beam.

Case II

ed at One End and Loaded with a Uniformly Distributed Load W

rmula (4)′

$$M_{max} = SI/12c$$

II; Chapter IX,

$$M_{max} = Wl/2$$

$$Wl/2 = SI/12c$$

e load in pounds is

$$W = SI/6cl \qquad (7)$$

$$I/c = 6Wl/S \qquad (7)'$$

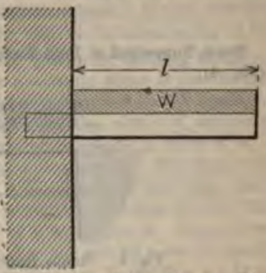


Fig. 2. Cantilever Beam. Distributed Load over Entire Span

2. What is the size of a can-
l I beam required to carry a uniformly distributed load of 150 lb per
ngth of 6 ft?

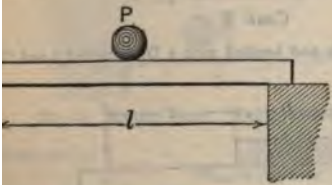
$W = 150 \times 6 = 900$ lb. Substituting in formula (7)′,

$$I/c = \frac{6 \times 900 \times 6}{16\,000} = 2.25$$

V, page 355, the nearest section-modulus to this is 1.9, which is
3-in 7.5-lb beam, the heaviest of that depth. However, as the
n beam, also, weighs 7.5 lb per ft it probably would be selected be-
s greater stiffness, although its section-modulus is 3, still greater
red.

Case III

pported at Both Ends and Loaded with a Concentrated Load at the
g. 3).



From Formula (4)′

$$M_{max} = SI/12c$$

From Case IV, Chapter IX,

$$M_{max} = Pl/4$$

Hence

$$Pl/4 = SI/12c$$

and the safe load in pounds is

$$P = SI/3cl \qquad (8)$$

and

$$I/c = 3Pl/S \qquad (8)'$$

Simple Beam. Load at Middle of Span

3. What steel I beam will safely support a concentrated load of
lied at the middle of a 15-ft span?

. $P = 7$ tons $= 14\,000$ lb. Substituting in formula (8)′,

$$I/c = \frac{3 \times 14\,000 \times 15}{16\,000} = 39.3$$

again *to Table IV, page 355, it is seen that a 12-in 35-lb* beam has

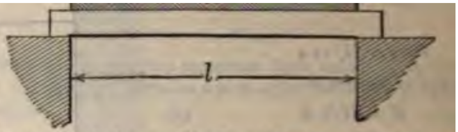


Fig. 4. Simple Beam. Distributed Load over Entire Span

From Formula (4)′

$$M_{\max} = SI/12c$$

From Case V, Chapter IX,

$$M_{\max} = Wl/8$$

Hence

$$Wl/8 = SI/12c$$

and the safe load in pounds is

$$W = 2SI/3cl$$

and

$$I/c = 3Wl/2S$$

Example 4. What steel **I** beam will safely carry a uniformly dis of 1 000 lb per ft over a span of 25 ft?

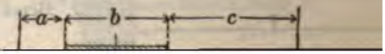
Solution. $W = wl = 1\,000 \times 25 = 25\,000$ lb. Substituting in F

$$I/c = \frac{3 \times 25\,000 \times 25}{2 \times 16\,000} = 58.6$$

From Table IV, page 354, the nearest section-modulus is 58.9, wh a 15-in 42-lb beam.

Case V

Beam Supported at Both Ends and Loaded with a Distributed Load of the Span (Fig. 5).



aximum bending moment as explained for Case VIII, Chapter IX, and ituting the value thus found in Formulas (3)′ or (5)′.

mple 5. What steel **I** beam will safely carry a uniformly distributed load oo lb per ft over part of the span, beginning at a point 5 ft from the left on and extending over a distance of 6 ft, the span of the beam being 18 ft?

ution. The first step is to find the point of maximum bending moment, is the point of no shear. Obviously the maximum shear is just at the of the reaction nearest the load, which in this case is the left reaction. nd the left reaction (see Chapter IX, page 324) the center of moments en at the right reaction and the equation of moments is $R_1 \times 18$ ft − lb × 6 ft) × 10 ft = 0. $18\,R_1 = 72\,000$ and $R_1 = 4\,000$ lb. The shear just e right of R_1 is therefore +4 000 lb which, if the weight of the beam itself considered, remains unchanged for every section of the beam between the eaction and the uniformly distributed load of 1 200 lb per ft. From there passing to the right, the shear is diminished at the rate of 1 200 lb per ft; it becomes zero, therefore, at a point 4 000 lb/1 200 lb per ft = 3.3 ft to ight of the 5-ft point. Hence the point of no shear and consequently the of maximum bending moment is at 5 ft + 3.3 ft, or 8.3 ft, from the left end. equation for the maximum bending moment at this point is, therefore,

$$\begin{aligned} M_{\max} &= 4\,000 \text{ lb} \times 8.3 \text{ ft} - (1\,200 \text{ lb} \times 3.3 \text{ ft}) \times 3.3/2 \text{ ft} \\ &= 33\,200 \text{ ft-lb} - 6\,574 \text{ ft-lb} = 26\,626 \text{ ft-lb, or } 319\,512 \text{ in-lb} \end{aligned}$$

m Formula (3), $I/c = 319\,512$ in-lb/16 000 lb per sq in = 20. From Table page 355, the nearest section-modulus corresponding to this is 20.4, that 9-in 25-lb beam. A 10-in 25-lb beam, however, being stronger and stiffer, ld probably be used. The 10-in 22 lb beam is what is termed a SUPPLE-TARY BEAM. (See page 353.)

Case VI

eam Supported at Both Ends and Loaded with a Concentrated Load, not at Middle (Fig. 6).

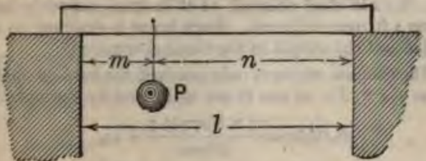


Fig. 6. Simple Beam. Concentrated Load at any Point

rom Formula (4)′,

$$M_{\max} = SI/12\,c$$

m Case VI, Chapter IX,

$$M_{\max} = Pmn/l$$

ce

$$Pmn/l = SI/12\,c$$

the safe load in pounds is

$$P = SIl/12\,cmn \tag{10}$$

$$I/c = 12\,Pmn/lS \tag{10$'$}$$

and *l* being *in feet*.

$$20 \times 16\,000$$

Table IV, page 354, the nearest value for the section-modul 1–1 is above 75.6, or 81.2 for a 15-in 60-lb beam. An 18-in 55-l a section-modulus of 88.4 would be used, unless conditions fix t as it weighs 5 lb per ft less, and being deeper is consequently stif

Case VII

Beam Supported at Both Ends and Loaded Symmetrically w Concentrated Loads (Fig. 7).

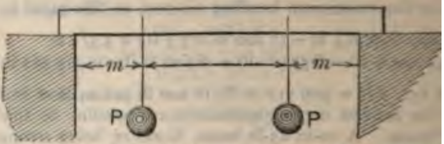


Fig. 7. Simple Beam. Equal Concentrated Loads Symmetrical

From Formula (4)′ and Case VII, Chapter IX, each of the safe l is

$$P = SI/12\,cm$$

and

$$I/c = 12\,Pm/S$$

Example 7. A 12-in steel channel, 12 ft in span, supports ha two 10-in beams 4 ft from each end. Each beam is designed to c What is the size and the weight of the channel required?

Solution. The channel supports only one-half the load on each $P = 8\,000$ lb, $m = 4$ ft, $S = 16\,000$ lb per sq in, and by Formula (

channel in the above example between the supports would be 300 lb, and P would be 8 100 lb, which would give a value for The factor of safety in the loads allowed is generally large enough light effect produced by the weight of the beam; but if the full is likely to be imposed on the beam, then allowance must be made of the beam itself.

Case VIII

orted at Both Ends and Loaded Symmetrically with Several Con- ds (Fig. 8).

it is necessary to compute the maximum bending moment in the portion the beam by Formulas (3)′ or (5)′.

A steel-beam girder is to be designed to support a brick wall, nd weighing 138 000 lb, over an opening 22 ft wide. The girder pport the ends in floor-beams Fig. 8, each ng 16 000 lb. ize and weight required?

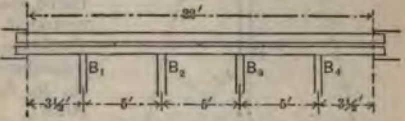


Fig. 8. Simple Beam. Several Concentrated Loads Symmetrically Placed

The first step allowance for f the girder. on the girder (neglecting the weight of the girder itself) = 138 000 lb (one-half the load on each beam) = 170 000 lb, or 85 tons. ch more than the heaviest single rolled beam will carry, it will be use a pair of beams and the load on each beam, therefore, will be onsidering for the present the entire load as uniformly distributed, ge 577, shows that to support 42.5 tons, or 85 000 lb, over a span res a 24-in 85-lb beam. The girder then will weigh between sup- 22 = 3 740 lb, or about 4 000 lb. This added to the weight of es, for the total distributed load, 142 000 lb. The next step is to maximum bending moment.

mulas given in Chapter IX the maximum bending moments for ads may be found as follows:

and girder (Case V, page 326),

$$M_{\max} = \frac{22 \times 142\,000}{8} = 390\,500 \text{ ft-lb}$$

B_1 (Case VI, page 327),

$$M_{1\max} = \frac{8\,000 \times 3\tfrac{1}{2} \times 18\tfrac{1}{2}}{22} = 23\,545 \text{ ft-lb}$$

B_2 (Case VI, page 327),

$$M_{2\max} = \frac{8\,000 \times 8\tfrac{1}{2} \times 13\tfrac{1}{2}}{22} = 41\,727 \text{ ft-lb}$$

being spaced symmetrically from the middle of the span, the bend- for B_3 and B_4 will be equal to those of B_2 and B_1 respectively. bending moments to a scale, in the manner explained for Figs. 17 330, the diagram shown *in Fig. 9 is obtained. The greatest bend- the ordinate M_2, which scales 486 500 ft-lb, or 5 838 000 in-lb.*

Note. Since the loads are symmetrically placed, this ordinate i middle point of the girder, but it is drawn to one side in the figure in to confuse it with the ordinate M, the maximum bending moment f formly distributed load. Substituting this value of M_x in formula (1),

$$I/c = \frac{5\,838\,000 \text{ in-lb}}{16\,000 \text{ lb per sq in}} = 365$$

the section-modulus for both beams, or 182.5 for one beam. From page 354, it is found that a 24-in 90-lb beam has a section-modulu and two 90-lb beams will jus The assumption of a uniform di of such a loading over every girder usually results in the se lighter beams than are indicate second solution, in which eac trated load is considered as n centrated at a point. The tw should be securely bolted toge separators near each connection B_1, B_2, B_3, B_4, and at each en girder.

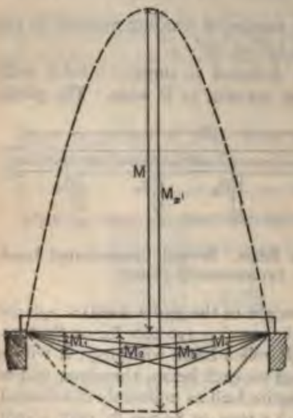


Fig. 9. Bending-moment Diagram for Beam Shown in Fig. 8.

A DOUBLE-BEAM GIRDER, ho not considered the best kind of use under this condition of loadin not good construction nor econ material. As a general rule m DERS should be used only when can be applied to the upper f both beams. Transferring a loa to the web of one beam, even is connected with the other means of separators, does not equal distribution of the loading. The author, therefore, recommend case a RIVETED BEAM GIRDER or a RIVETED PLATE GIRDER. The meth indicated applies to any method of loading, the only difference in culation being in the determination of the maximum bending momen

Inclined Beams. The strength of beams inclined to the horizont computed, with sufficient accuracy for most purposes, by using the given for horizontal beams, and taking the HORIZONTAL PROJECTIONS of as the spans.

3. Steel Beams and Girders*

Materials Used for Beams. Practically the only materials used tural work for beams, at the present day, are wood, steel and reinforced As wooden beams are always rectangular in cross-section, the general used in this chapter can be much simplified by substituting for I/c it terms of the breadth and depth of the beam. Formulas for wooden b therefore be found in Chapter XVI. Cast iron, also, is occasionally beams or lintels, but as this material is much stronger in resisting co than tension, the beam must be of a special shape in order to use th to advantage. The strength of cast-iron beams is therefore conside

---

* For the deflection of steel beams, see Chapter XVIII.

94.6	83.0	28.7	21.6

eepest Beams, the Strongest, Stiffest and Most Economical. NGTH of a wooden or steel beam of rectangular cross-section varies as E OF THE DEPTH, directly as the breadth and inversely as the length, TIFFNESS varies directly as the CUBE OF THE DEPTH, directly as the nd inversely as the cube of its length; hence the deeper beam will greater strength and stiffness in proportion to its sectional area. With these relations do not hold strictly, because of the variation in the the cross-sections, but they are approximately true. It therefore at, for any given span, it is more economical in floors, where other s will permit, to use deep beams spaced farther apart or to use one deep place of two shallower beams. Thus if a distributed load of 39 tons upported over a span of 16 ft, one 20-in 65-lb beam, two 15-in 42-lb three 12-in 40-lb beams, could be used; but the 20-in beam would y 1 105 lb, allowing for 6-in bearings, as compared with 1 428 lb for beams and 2 040 lb for the 12-in beams, and the bolts and separators saved.

nd Heavy Steel Beams. LIGHT BEAMS are more economical than ams OF THE SAME DEPTH, except when the span is so short that the is governed by the resistance of the web to buckling, in which case the AMS are the more economical.

um Safe Loads for Steel Beams. All loaded beams are, in general, o three kinds of stresses. The most destructive are generally those e BENDING MOMENTS, and have already been considered. The second those which tend to SHEAR a beam, or to make one part slide on vertically. (See paragraph on Shearing-Stresses in Steel Beams and page 567.) These stresses, however, seldom need to be considered the case of riveted girders and short beams with very thick webs. kind of stress is that which tends to cause the web of a beam to and in a steel beam over a span very short in proportion to the the beam, the resistance of the web to buckling generally determines

For several concentrated loads or for a combination of distribute
trated loads it will be necessary to use the methods previously ex
Case VIII, page 563.

Use of Tables for Concentrated Loads. To use any of
tables for CONCENTRATED LOADS, find the equivalent distributed l
plying the concentrated load by the factor given in Table IV, p
then use the beam having a safe load equal to the load thus found.

In addition to the conversion-factors in that table the followin
found convenient:

For two equal loads applied at one-third the span from each end,
load by 2⅔.

For two equal loads applied at one-fourth the span from each
one load by 2.

For a beam fixed at one end, and loaded at the other, multiply b

For a beam fixed at one end, and uniformly loaded over the
multiply by 4.

Unusual Conditions of Loading of Beams.* It is assume
that the loads are applied normal to the axis 1–1 as shown in the
properties of sections in Chapter X, and that the beam deflects ve
PLANE OF BENDING ONLY. If the conditions of loading involve
tion of forces outside this plane of loading, the allowable safe l
determined from the general theory of flexure in accordance with
application of the load and its character. This applies particular
METRICAL SECTIONS, such as angles, which should be used under
tions of loading where the section can deflect vertically only, bei
cured against LATERAL DEFLECTION or twisting throughout the
In all such cases of eccentric loading, the actual safe loads would be
lower than the tabulated safe loads, which have been based up
favorable conditions of loading.

Vertical Deflection of Steel Beams.* In the case of beams
carry plastered ceilings, experience indicates that the VERTICAL
to avoid cracking the plaster, should be limited to not more tha

ıms are loaded with heavy loads concentrated near the supports,
ns of short span are loaded with uniformly distributed loads to their
capacity as regards flexure, the bending moments may be small
n with the reactions at the supports, and the beams may fail along
ırface as a result of LONGITUDINAL SHEARING-STRESSES, or they may
result of the combined longitudinal and vertical web-stresses.
ns the safe shearing or buckling strength of the web rather than
e of the flanges to bending-stresses may limit the carrying capacity

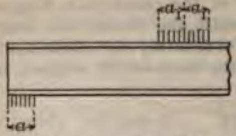
Values of Beam-Webs.* The VERTICAL SHEARING-STRESSES or
compressive components of the web-stresses may under some con-
d the safe resistance of the beam to BUCKLING, and there remains
y that a web or web-plate, which is amply secure against the safe
r of 10 000 lb per sq in, will not be of sufficient strength when con-
column. In such cases provision must be made for security against
her by stiffeners or by an increased thickness of the web or web-
the determining conditions for web-buckling of steel beams in gril-
on direct compression, see page 183.)

s of Web-Buckling of Steel Beams. There are two conditions
LING (see, also, foot-note for paragraphs relating to Tables II and

ırt of the beam bearing on the support is subject to DIRECT COM-
d the web over this part must be capable of resisting it. If this
nall the end of the beam will fail, as a column, causing the web to
is therefore necessary to calculate the required length of the bear-

eam throughout its length between the supports, or in case of a
am, from its end to the support, is subject to SHEAR. It is gen-
ed that the shear develops stresses of TENSION and COMPRESSION
that these stresses act at right-angles to each other in the plane of
at an angle of 45° with the neutral surface of the beam; and that
AL STRESSES are equal in magnitude or intensity to the VERTICAL

beams with unsupported webs, and from these experiments the fo
mulas* have been deduced:



Safe end-reaction $R = S_b \times t\Big($

Safe interior load $P = 2 S_b \times t\Big($

In these formulas, R is the end-reaction, P the concentrated load
thickness, d the depth of the beam, a_1 half the distance over which
trated load is applied and a the whole distance over which the en
applied; while S_b is the SAFE RESISTANCE OF THE WEB TO BUCKLIN
per square inch, by the straight-line formula

$$S_c = 19\,000 - 100\,d/2\,r$$

$d/2 = l$ in the column-formula †. The first formula is general a
any condition of loading. The second formula covers the case of
concentrated at the middle of a span; it can be extended to cover
concentrated loads provided the sum of the distances a_1 is not less t

Tables II ‡ and III ‡ give for beams and channels with
webs:

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* These formulas, in order to satisfy the first condition, are used in the
panion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

† This is the column-formula used by the American Bridge Company and
Pocket Companion, S_c being the allowable COMPRESSIVE UNIT STRESS i
square inch within the usual limits of l/r

‡ In regard to the shearing of steel beams, allowable web-shears, etc.,
example (see Example 15, this chapter, and on pages 182 and 183 of C
42 000 lb per sq in for a 12-in, 31½-lb **I** beam, given in Table II, page 57
Carnegie's Pocket Companion, is based on the allowed direct shear without
condition of web-crippling. That is, the 42 000 lb is determined by takin
the web, 0.35 × 12 = 4.2 sq in and multiplying it by 10 000 lb per sq in,
value there used for the safe unit shearing-stress.

The beam is therefore calculated as being good for 42 000 lb SHEAR, but
to make a further investigation to ascertain whether the stresses due to sh
the web of the beam to buckle. As stated in the paragraph on page 567, on
Values of Beam-Webs there are two conditions of web-buckling or web-crip

In the case of a plate girder the end-stiffeners provide for the first cond
intermediate stiffeners for the second condition. The web itself may then
for its full shearing value. In the case of beams, however, it is not general
to use stiffeners, so that the web alone must meet every condition.

The Carnegie Pocket Companion gives a formula, reproduced in the pr
graph, and gives the derived lengths of bearings in Tables II and III, to sa
condition. Some of the formulas used in the manufacturers' handbooks,

may be calculated by the formula,

$$V = \frac{12\,000\,dt}{1 + \dfrac{h^2}{1\,500\,t^2}}$$

= the maximum safe web-shear in pounds; d = the depth of the
the thickness of the web; and h = the height between the flange-
e Example 15, this chapter and also example on pages 182 and 183.)
ition to these data which have to do with the MAXIMUM LOADS on beams
els as computed from the WEB-RESISTANCE, Tables II and III give,
AXIMUM BENDING MOMENTS in foot-pounds, obtained by the multi-
f the SECTION-MODULUS of each section by the allowed FIBER-STRESS
b per sq in and the division of the product by 12 in order to reduce
ound basis. These maximum bending moments may be used on
instead of the table of properties to ascertain the proper size of a
e used in any particular instance."

ch V = the maximum safe web-shear in pounds; d = the depth of the beam;
ness of web; and h = the distance between the flange-fillets.

noted that the length of the element in compression on the 45° line is $h\sqrt{2}$,
square of this length is $2\,h^2$. It is this value, $2\,h^2$, that is substituted for l^2
n-formula used by the Cambria Steel Company in deducing its formula for
on web-buckling. The tensile stress, however, tends to keep the compressive
buckling the web, and for this reason the Passaic and Bethlehem engineers
re liberal value of $3\,000\,t^2$ instead of $1\,500\,t^2$. The Passaic Steel Company,
ed the more conservative unit value of 10 000 lb, reduced, instead of the
ed by the others. The Passaic and Cambria formulas give about the same
-in, 31½-lb **I** beam by the former having a safe shear of 33 352 lb and by the
8 lb.

ic Steel Company is no longer in existence and their handbook is out of print.
em Steel Company's handbook has tables for Bethlehem shapes only. If,
no table of maximum shears of beams, based on web-crippling, is at hand, it
that the values may be determined from the formula,

$$V = \frac{12\,000\,dt}{}$$

illustration of the use of these tables let it be required to det
size and weight of an **I** beam to carry safely a uniformly d
34 000 lb over a span of 20 ft, the weight of the beam not bein

Solution. From Table IV, page 579, a 15-in 50-lb beam wi
The weight of this beam is 50 lb × 20 ft = 1 000 lb, making
supported of 35 000 lb. This is so little in excess of the safe lo
need not be considered. Had the difference been more, h
heavier beam should be used.

Example 10. Direct Bending from a Concentrated Load.
use of the tables to determine the size and weight of beams
concentrated loads, Examples 10 and 11 are given. What **I** be
will safely support 8 000 lb, concentrated at a point 5 ft from

Solution. The distance 5 ft is one-third of the span, an
factor for this (Table IV, page 632) is 1.78. The equivale
tributed load, therefore, is 8 000 × 1.78 = 14 240 lb, and fro
581, a 9-in 25-lb **I** beam will carry 14 500 lb for a span of 1
answer the purpose.

Example 11. Direct Bending from Two Equal Concentrat
I beam, 15 ft in span, will safely support two equal concentrat
lb each, applied 5 ft from each end?

Solution. The distance 5 ft is one-third the span, but the
case is 2⅔ (page 632). Hence, the equivalent uniformly di
6 000 × 2⅔ = 16 000 lb and the beam required (Table IV, pa
25-lb, **I** beam which will carry 17 400 lb. The same resul
using Formula (11)′, page 562. This formula is, $I/c = 12\,Pm/$
$I/c = 12 \times 6\,000 \times 5/16\,000 = 22.5$. The nearest section-modu
that of a 10-in 25-lb **I** beam.

Example 12. Maximum Bending Moment from a Distribute
of the Span. The beam in Example 5, Case V, page 561, has a
ing moment of 26 626 ft-lb. What beam is required?

Solution. The nearest bending moment to this in the first nd by the formula given on page 309 for the second condition of web-

$$V = \frac{12\,000\,dt}{1 + \frac{h^2}{1\,500\,t^2}}$$

dimensions of structural beams (see Carnegie's Pocket Companion,
h Edition, page 64) the thickness t of the web of a 12-in, 31.5-lb **I** beam
he depth of the beam is $d = 12$ in and h, the distance between flange-
762 in. Substituting these values in the formula,

$$\frac{00 \times 12 \times 0.35}{\frac{9.762^2}{1\,500 \times 0.35^2}} = \frac{50\,400}{1 + \frac{95.296644}{1\,500 \times 0.1225}} = \frac{50\,400}{1 + \frac{95.296644}{183.75}} = \frac{50\,400}{\frac{279.046644}{183.75}}$$

$$= \frac{50\,400 \times 183.75}{279.046644} = \frac{9\,261\,000}{279.046644} = 33\,188, \text{ or about } 33\,190 \text{ lb}$$

s less than the allowable web-shear of 42 000 lb given in the tables,
is to be taken of the web-buckling from the second condition men-
he preceding pages, a larger or heavier beam should be used or the
ed, so that the maximum shear will not exceed 33 190 lb. (For de-
conditions for web-buckling of steel beams in grillages, based on
pression, see page 183.)

16. **Safe End-Reactions for Web-Buckling.** In Example 8, page
o 24-in 90-lb **I** beams carry 170 000 lb + (4 000 lb, the weight of the
74 000, lb or 87 000 lb for each beam. Assuming that they rest
brackets riveted to columns at each end of the span, are the end-
xcessive?

Since the loading is symmetrical, each reaction for each beam is
e total load on each beam, or 43 500 lb. From the last column in
age 574, the maximum end-reaction R, for a 24-in 90-lb beam, is
Hence, the beam is safe as far as the compression from the end-
concerned.

for the rafter, set with the long legs vertical and ½ in apart.
dimensions of the angles, the strut being braced laterally?

Solution. (1) By Formula (8)′, $I/c = 3 \times 2\,800 \times 8.5/12\,00$
pair of angles, or 2.98 for each angle. (See note at end of this e
Table XI, page 363, the nearest value to this with reference to
3.0, the section-modulus for a 5 by 3½ by ½-in angle. The sec
angle is 4 sq in and of two angles, 8 sq in.

(1) From Table XVI, page 371, the least r for a pair of 5
angles, which would be about the axis 1–1, since the strut is
is about 1.58 (between 1.55 and 1.61). Then the slenderness-
6 in/1.58 in = 102 in/1.58 in = 64.5. From column VIII, T
493, $S = 9\,250$ lb per sq in. Hence, 30 000 lb/9 250 lb per sq
approximately.

(3) The section-area required, therefore, is $8 + 3.24 = 11.24$ sq
Table XI, page 363, is about equivalent to that of two 5 b
angles. As the section-area in both calculations exceeds that a
no allowance for the weight of the angles need be made.

Note. Because of the increase in the tendency of the strut to
by the combined stresses of flexure and compression, lower valu
than in the cases of simple flexure, or of simple compression.

Tie-Beams. Steel beams subject to combined tensile and tra
should be calculated in a way similar to that explained above f
The section necessary to resist the transverse stress should be f
the section-area necessary to resist the tensile stress, and the two

Example 18. Tie-Beam. Combined Bending and Tension.
tie-beam, 10 ft between joints, supports a load of 6 000 lb at the
the same time is under a tensile stress of 84 000 lb. It is propo
steel channels for the tie-beam. What size and weight are r
channels?

Solution. A load of 6 000 lb applied at the middle of a bea
effect as a load of 12 000 lb uniformly distributed, or 6 000 lb fo
From Table V, page 584, a 7-in, 9.75-lb channel will be requir

o = 1.7. From Table VIII, page 359, the I/c about the axis 2–2 g to this is that of a 12-in, 20.5-lb channel.

o. **Rectangular Steel Bar with Long Side Vertical.** In a suspended, ling it is proposed to use 2 by ⅜-in steel bars, 4 ft or 48 in long, to ister. What is the safe load each bar will support, if set with the tical?

From Table I, page 346, the I for a 2 by ⅜-in bar is 0.250. $c =$ depth = 1 in. $I/c = 0.250/1 = 0.250$. Also, from Formula (2)′, $M_{max} = SI/c$. Substituting, $M_{max} = 16\,000 \times 0.250 = 4\,000$ in-lb. ase V, page 326, $M_{max} = Wl/8$, and hence, $4\,000 = W \times 48/8 =$ $= 4\,000/6 = 666$ lb.

Table II.* † Maximum Bending Moments and Web-Resistances of I

M_{max} Maximum bending moment ft-lb	d Depth of beam in	w Weight per lin ft lb	t Thickness of web in	V Allowable web-shear lb	S_b† Allowable buckling resistance lb per sq in	a Minimum end-bearing in
285 300	27	83.0	0.424	114 480	7 970	27.1
328 390	24	115.0	0.750	180 000	13 460	11.8
320 390		110.0	0.688	165 120	12 960	12.5
312 390		105.0	0.625	150 000	12 350	13.4
264 400		100.0	0.754	180 960	13 490	11.8
256 560		95.0	0.693	166 320	13 000	12.5
248 710		90.0	0.631	151 440	12 410	13.3
240 870		85.0	0.570	136 800	11 710	14.5
231 920		80.0	0.500	120 000	10 690	16.5
214 220		69.5	0.390	93 600	8 340	22.8
155 880	21	57.5	0.357	74 970	8 820	18.6
220 750	20	100.0	0.884	176 800	15 080	8.3
214 210		95.0	0.810	162 000	14 720	8.6
207 680		90.0	0.737	147 400	14 300	9.0
201 140		85.0	0.663	132 600	13 780	9.5
195 510		80.0	0.600	120 000	13 230	10.1
169 170		75.0	0.649	129 800	13 660	9.6
162 640		70.0	0.575	115 000	12 980	10.4
155 930		65.0	0.500	100 000	12 080	11.6
186 720	18	90.0	0.807	145 260	15 140	7.4
180 840		85.0	0.725	130 500	14 700	7.7
174 960		80.0	0.644	115 920	14 160	8.2
169 080		75.0	0.562	101 160	13 450	8.9
136 480		70.0	0.719	129 420	14 670	7.8
130 590		65.0	0.637	114 660	14 110	8.3
124 710		60.0	0.555	99 900	13 380	9.0
117 860		55.0	0.460	82 800	12 220	10.2
108 620		46.0	0.322	57 960	9 320	14.8
122 890	15	75.0	0.882	132 300	16 050	5.6
117 980		70.0	0.784	117 600	15 690	5.8
113 080		65.0	0.686	102 900	15 210	6.1
108 270		60.0	0.590	88 500	14 600	6.5
90 850		55.0	0.656	98 400	15 040	6.2
85 940		50.0	0.558	83 700	14 340	6.7
81 040		45.0	0.460	69 000	13 350	7.5
78 530		42.0	0.410	61 500	12 670	8.1
72 020		36.0	0.289	43 350	10 010	11.2

V is computed at 10 000 lb per sq in of gross area of web-section.

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

† See, also, foot-note on page 568, with paragraphs relating to this table and to
III, and paragraphs on page 567, relating to web-buckling of steel beams.
page 183.

† (Continued). Maximum Bending Moments and Web-Resistances of I Beams

d	w	t	V	S_b†	a	R
Depth of beam	Weight per lin ft	Thickness of web	Allowable web-shear	Allowable buckling resistance	Minimum end-bearing	End-reaction $a=3\frac{1}{2}$ in
in	lb	in	lb	lb per sq in	in	lb
12	55.0	0.821	98 520	16 470	4.3	87 890
	50.0	0.699	83 880	16 030	4.5	72 830
	45.0	0.576	69 120	15 390	4.8	57 620
	40.0	0.460	55 200	14 480	5.3	43 300
	35.0	0.436	52 320	14 230	5.4	40 330
	31.5	0.350	42 000	13 060	6.2	29 710
	27.5	0.255	30 600	10 850	8.1	17 990
10	40.0	0.749	74 900	16 690	3.5	75 010
	35.0	0.602	60 200	16 120	3.7	58 220
	30.0	0.455	45 500	15 190	4.1	41 470
	25.0	0.310	31 000	13 410	5.0	24 940
	22.0	0.232	23 200	11 540	6.2	16 060
9	35.0	0.732	65 880	16 870	3.1	71 010
	30.0	0.569	51 210	16 260	3.3	53 200
	25.0	0.406	36 540	15 160	3.7	35 390
	21.0	0.290	26 100	13 620	4.4	22 710
8	25.5	0.541	43 280	16 440	2.9	48 920
	23.0	0.449	35 920	15 910	3.0	39 290
	20.5	0.357	28 560	15 120	3.3	29 690
	18.0	0.270	21 600	13 870	3.8	20 600
	17.5	0.210	16 800	12 400	4.5	14 320
7	20.0	0.458	32 060	16 350	2.5	39 310
	17.5	0.353	24 710	15 570	2.7	28 850
	15.0	0.250	17 500	14 150	3.2	18 580
9	17.25	0.475	28 500	16 810	2.1	39 930
	14.75	0.352	21 120	16 050	2.2	28 250
	12.25	0.230	13 800	14 480	2.6	16 650
5	17.0	0.380	19 000	16 720	1.7	30 180
	14.75	0.504	25 200	17 280	1.6	41 370
	12.25	0.357	17 850	16 580	1.8	28 120
	9.75	0.210	10 500	14 870	2.1	14 830
4	10.5	0.410	16 400	17 310	1.3	31 940
	9.5	0.337	13 480	16 940	1.4	25 690
	8.5	0.263	10 520	16 360	1.4	19 360
	7.5	0.190	7 600	15 360	1.6	13 130
3	7.5	0.361	10 830	17 560	1.0	26 940
	6.5	0.263	7 890	17 020	1.0	19 020
	5.5	0.170	5 100	15 950	1.1	11 530

V is computed at 10 000 lb per sq in of gross area of web-section.

Pocket Companion, 1915 Edition, *Carnegie Steel Company, Pittsburgh, Pa.*
so, foot-*note on page 568, with paragraphs relating to this table and to* Table
ragraphs on page 567, relating to web-buckling of steel beams. See, also,

Table III.*† Maximum Bending Moments and Web-Resistances of

M_{max}	d	w	t	V	S_b†	s
Maximum bending moment	Depth of channel	Weight per lin ft	Thickness of web	Allowable web-shear	Allowable buckling resistance	Minimum end-bearing
ft-lb	in	lb	in	lb	lb per sq in	in
76 490	15	55.0	0.818	122 700	15 820	5.7
71 590		50.0	0.720	108 000	15 390	6.0
66 680		45.0	0.622	93 300	14 820	6.4
61 780		40.0	0.524	78 600	14 040	6.9
56 880		35.0	0.426	63 900	12 900	7.9
55 570		33.0	0.400	60 000	12 510	8.2
64 360	13	50.0	0.791	102 830	16 150	4.8
60 110		45.0	0.678	88 140	15 680	5.0
55 870		40.0	0.565	73 450	15 020	5.4
53 320		37.0	0.497	64 610	14 470	5.7
51 620		35.0	0.452	58 760	14 020	6.0
48 740		32.0	0.375	48 750	13 000	6.8
43 760	12	40.0	0.758	90 960	16 260	4.4
39 840		35.0	0.636	76 320	15 730	4.6
35 920		30.0	0.513	61 560	14 950	5.0
32 000		25.0	0.390	46 800	13 670	5.8
28 470		20.5	0.280	33 600	11 570	7.4
30 800	10	35.0	0.823	82 300	16 900	3.4
27 530		30.0	0.676	67 600	16 440	3.6
24 260		25.0	0.529	52 900	15 730	3.9
20 990		20.0	0.382	38 200	14 470	4.4
17 840		15.0	0.240	24 000	11 780	6.0
20 950	9	25.0	0.615	55 350	16 470	3.2
18 010		20.0	0.452	40 680	15 550	3.5
15 070		15.0	0.288	25 920	13 590	4.4
14 020		13.25	0.230	20 700	12 220	5.1
15 920	8	21.25	0.582	46 560	16 620	2.8
14 610		18.75	0.490	39 200	16 170	2.9
13 310		16.25	0.399	31 920	15 530	3.2
12 000		13.75	0.307	24 560	14 490	3.5
10 770		11.25	0.220	17 600	12 700	4.3
12 640	7	19.75	0.633	44 310	17 090	2.3
11 490		17.25	0.528	36 960	16 700	2.4
10 350		14.75	0.423	29 610	16 130	2.6
9 210		12.25	0.318	22 260	15 190	2.9
8 030		9.75	0.210	14 700	13 230	3.5
8 680	6	15.5	0.563	33 780	17 150	2.0
7 700		13.0	0.440	26 400	16 640	2.1
6 720		10.5	0.318	19 080	15 730	2.3
5 780		8.0	0.200	12 000	13 810	2.8
5 550	5	11.5	0.477	23 850	17 180	1.7
4 730		9.0	0.330	16 500	16 380	1.8
3 960		6.5	0.190	9 500	14 450	2.2
3 050	4	7.25	0.325	13 000	16 870	1.4
2 790		6.25	0.252	10 080	16 250	1.5
2 530		5.25	0.180	7 200	15 150	1.6
1 840	3	6.0	0.362	10 860	17 560	1.0
1 640		5.0	0.264	7 920	17 030	1.0
1 450		4.0	0.170	5 100	15 940	1.1

V is computed at 10 000 lb per sq in of gross area of web-section.

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh

† See, also, foot-note on page 568, with paragraphs relating to this table and to III, and paragraphs on page 567, relating to web-buckling of steel beams. See, page 183.

le IV.* Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress, 16 000 lb per sq in

an, ft	Depth and weight of sections											Coefficient of deflection
	27-in	24-in									21-in	
	83 lb	115 lb	110 lb	105 lb	100 lb	95 lb	90 lb	85 lb	80 lb	69½ lb	57½ lb	
....	...	...	...	...	371.9	...	...	...	...	...	...	....
6	...	...	...	...	352.5	332.6	302.9	...	...	...	...	0.60
7	...	360.0	330.2	...	302.2	293.2	284.2	273.6	240.0	...	...	0.81
8	...	328.4	320.4	300.0	264.4	256.6	248.7	240.9	231.9	...	150.0	1.06
9	229.0	291.9	284.8	277.7	235.0	228.0	221.1	214.1	206.1	187.2	138.6	1.34
0	228.2	262.7	256.3	249.9	211.5	205.2	199.0	192.7	185.5	171.4	124.7	1.66
1	207.5	238.8	233.0	227.2	192.3	186.6	180.9	175.2	168.7	155.8	113.4	2.00
2	190.2	218.9	213.6	208.3	176.3	171.0	165.8	160.6	154.6	142.8	103.9	2.38
3	175.6	202.1	197.2	192.2	162.7	157.9	153.1	148.2	142.7	131.8	95.9	2.80
4	163.0	187.7	183.1	178.5	151.1	146.6	142.1	137.6	132.5	122.4	89.1	3.24
5	152.2	175.1	170.9	166.6	141.0	136.8	132.6	128.5	123.7	114.3	83.1	3.72
6	142.6	164.2	160.2	156.2	132.2	128.3	124.4	120.4	116.0	107.1	77.9	4.24
7	134.3	154.5	150.8	147.0	124.4	120.7	117.0	113.4	109.1	100.8	73.4	4.78
8	126.8	146.0	142.4	138.8	117.5	114.0	110.5	107.1	103.1	95.2	69.3	5.36
9	120.1	138.3	134.9	131.5	111.3	108.0	104.7	101.4	97.6	90.2	65.6	5.98
0	114.1	131.4	128.2	125.0	105.8	102.6	99.5	96.3	92.8	85.7	62.4	6.62
1	108.7	125.1	122.1	119.0	100.7	97.7	94.7	91.8	88.3	81.6	59.4	7.30
2	103.7	119.4	116.5	113.6	96.1	93.3	90.4	87.6	84.3	77.9	56.7	8.01
3	99.2	114.2	111.4	108.7	92.0	89.2	86.5	83.8	80.7	74.5	54.2	8.76
4	95.1	109.5	106.8	104.1	88.1	85.5	82.9	80.3	77.3	71.4	52.0	9.53
5	91.3	105.1	102.5	100.0	84.6	82.1	79.6	77.1	74.2	68.6	49.9	10.35
6	87.8	101.0	98.6	96.1	81.4	78.9	76.5	74.1	71.4	65.9	48.0	11.19
7	84.5	97.3	94.9	92.6	78.3	76.0	73.7	71.4	68.7	63.5	46.2	12.07
8	81.5	93.8	91.5	89.3	75.5	73.3	71.1	68.8	66.3	61.2	44.5	12.98
9	78.7	90.6	88.4	86.2	72.9	70.8	68.6	66.4	64.0	59.1	43.0	13.92
0	76.1	87.6	85.4	83.3	70.5	68.4	66.3	64.2	61.8	57.1	41.6	14.90
1	73.6	84.7	82.7	80.6	68.2	66.2	64.2	62.2	59.8	55.3	40.2	15.91
2	71.3	82.1	80.1	78.1	66.1	64.1	62.2	60.2	58.0	53.6	39.0	16.95
3	69.2	79.6	77.7	75.7	64.1	62.2	60.3	58.4	56.2	51.9	37.8	18.03
4	67.1	77.3	75.4	73.5	62.2	60.4	58.5	56.7	54.6	50.4	36.7	19.13
5	65.2	75.1	73.2	71.4	60.4	58.6	56.8	55.1	53.0	49.0	35.6	20.28
6	63.4	73.0	71.2	69.4	58.8	57.0	55.3	53.5	51.5	47.6	34.6	21.45
7	61.7	71.0	69.3	67.5	57.2	55.5	53.8	52.1	50.1	46.3	33.7	22.66
8	60.1	69.1	67.5	65.8	55.7	54.0	52.4	50.7	48.8	45.1	32.8	23.90
9	58.5	67.4	65.7	64.1	54.2	52.6	51.0	49.4	47.6	43.9	32.0	25.18
0	57.1	65.7	64.1	62.5	52.9	51.3	49.7	48.2	46.4	42.8	31.2	26.48
1	55.7	64.1	62.5	61.0	51.6	50.1	48.5	47.0	45.3	41.8	30.4	27.82
2	54.3	62.6	61.0	59.5	50.4	48.9	47.4	45.9	44.2	40.8	29.7	29.20
3	53.1	61.1	59.6	58.1	49.2	47.7	46.3	44.8	43.1	39.9	29.0	30,60
4	51.9	59.7	58.3	56.8	48.1	46.6	45.2	43.8	42.2	38.9	28.3	32.04
5	50.7	58.4	57.0	55.5	47.0	45.6	44.2	42.8	41.2	38.1	...	33.52
6	49.6	57.1	55.7	54.3	46.0	44.6	43.3	41.9	40.3	37.3	...	35.02
7	48.6	55.9	54.1	53.2	45.0	43.7	42.3	41.0	39.5	36.5	...	36.56
8	47.5	54.7	53.4	52.1	44.1	42.8	41.5	40.1	38.7	35.7	...	38.14
9	46.6	53.6	52.3	51.0	43.2	41.9	40.6	39.3	37.9	35.0	...	39.74
0	45.6	52.5	51.3	50.0	42.3	41.0	39.8	38.5	37.1	34.3	...	41.38

Loads above the upper heavy lines will cause maximum allowable shears in bs. See, also, paragraphs in text and foot-note with same, page 567, relating to b-buckling in beams

Loads below the lower broken lines *will cause excessive deflections*

From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table IV* (Continued). Safe Uniform Loads in Units of 1 000
for Steel I Beams

Maximum bending stress, 16 000 lb per sq in

Span, ft	Depth and weight of sections										
	20-in								18-in		
	100 lb	95 lb	90 lb	85 lb	80 lb	75 lb	70 lb	65 lb	90 lb	85 lb	80 lb
....	353.6	...	...	...	...	...	...	...	...		
5	353.2	...	...	...	...	...	...	...	...	...	
....	....	324.0	294.8	...	...	259.6	230.0	...	290.5	261.0	...
6	294.3	285.6	276.9	265.2	240.0	225.6	216.8	200.0	249.0	241.1	211.8
7	252.3	244.8	237.7	229.9	223.4	193.3	185.9	178.2	213.4	206.7	200.0
8	220.7	214.2	207.7	201.1	195.5	169.2	162.6	155.9	186.7	180.8	175.0
9	196.2	190.4	184.6	178.8	173.8	150.4	144.6	138.6	166.0	160.7	155.5
10	176.6	171.4	166.1	160.9	156.4	135.3	130.1	124.7	149.4	144.7	140.0
11	160.5	155.8	151.0	146.3	142.2	123.0	118.3	113.4	135.8	131.5	127.2
12	147.2	142.8	138.5	134.1	130.3	112.8	108.4	104.0	124.5	120.6	116.6
13	135.8	131.8	127.8	123.8	120.3	104.1	100.1	96.0	114.9	111.3	107.7
14	126.1	122.4	118.7	114.9	111.7	96.7	92.9	89.1	106.7	103.3	100.0
15	117.7	114.2	110.8	107.3	104.3	90.2	86.7	83.2	99.6	96.4	93.3
16	110.4	107.1	103.8	100.6	97.7	84.6	81.3	78.0	93.4	90.4	87.5
17	103.9	100.8	97.7	94.1	92.0	79.6	76.5	73.4	87.9	85.1	82.3
18	98.1	95.2	92.3	89.4	86.9	76.3	72.3	69.3	83.0	80.4	77.8
19	92.9	90.2	87.4	84.7	82.3	71.2	68.5	65.7	78.6	76.1	73.7
20	88.3	85.7	83.1	80.5	78.2	67.7	65.1	62.4	74.7	72.3	70.0
21	84.1	81.6	79.1	76.6	74.5	64.4	62.0	59.4	71.1	68.9	66.7
22	80.3	77.9	75.5	73.1	71.1	61.5	59.1	56.7	67.9	65.8	63.6
23	76.8	74.5	72.2	70.0	68.0	58.8	56.6	54.2	64.9	62.9	60.9
24	73.6	71.4	69.2	67.0	65.2	56.4	54.2	52.0	62.2	60.3	58.3
25	70.6	68.5	66.5	64.4	62.6	54.1	52.0	49.9	59.8	57.9	56.0
26	67.9	65.9	63.9	61.9	60.2	52.1	50.0	48.0	57.5	55.6	53.8
27	65.4	63.5	61.5	59.6	57.9	50.1	48.2	46.2	55.3	53.6	51.8
28	63.1	61.2	59.3	57.5	55.9	48.3	46.5	44.6	53.3	51.7	50.0
29	60.9	59.1	57.3	55.5	53.9	46.7	44.9	43.0	51.5	49.9	48.3
30	58.9	57.1	55.4	53.6	52.1	45.1	43.4	41.6	49.8	48.2	46.7
31	57.0	55.3	53.6	51.9	50.5	43.7	42.0	40.2	48.2	46.7	45.2
32	55.2	53.6	51.9	50.3	48.9	42.3	40.7	39.0	46.7	45.2	43.7
33	53.5	51.9	50.4	48.8	47.4	41.0	39.4	37.8	45.3	43.8	42.4
34	51.9	50.4	48.9	47.3	46.0	39.8	38.3	36.7	43.9	42.6	41.2
35	50.5	49.0	47.5	46.0	44.7	38.7	37.2	35.6	42.7	41.3	40.0
36	49.1	47.6	46.2	44.7	43.4	37.6	36.1	34.7	41.5	40.2	38.9
37	47.7	46.3	44.9	43.5	42.3	36.6	35.2	33.7	40.4	39.1	37.8
38	46.5	45.1	43.7	42.3	41.2	35.6	34.2	32.8	39.3	38.1	36.8
39	45.3	43.9	42.6	41.3	40.1	34.7	33.4	32.0	...	...	...
40	44.1	42.8	41.5	40.2	39.1	33.8	32.5	31.2	...	...	...
41	43.1	41.8	40.5	39.2	38.1	33.0	31.7	30.4	...	...	...
42	42.0	40.8	39.6	38.3	37.2	32.2	31.0	29.7	...	...	...

Loads above the upper heavy lines will cause maximum allowable
webs. See, also, paragraphs in text and foot-note with same, page 567,
to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

* (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress, 16 000 lb per sq in

Depth and weight of sections												Coefficient of deflection
18-in				15-in								
65 lb	60 lb	55 lb	46 lb	75 lb	70 lb	65 lb	60 lb	55 lb	50 lb	45 lb	42 lb	
...	...	...	...	264.6	...	...	...	196.8	...	...	...	...
9.3	199.8	...	...	245.8	235.2	205.8	177.0	181.7	167.4	138.0	...	0.27
8.9	199.5	...	...	196.6	188.8	180.9	173.2	145.4	137.5	129.7	...	0.41
...	...	165.6	...	...	...	...	...	...	...	...	123.0	...
4.1	166.3	157.1	...	163.8	157.3	150.8	144.4	121.1	114.6	108.1	104.7	0.60
9.2	142.5	134.7	115.9	140.4	134.8	129.2	123.7	103.8	98.2	92.6	89.8	0.81
0.6	124.7	117.9	108.6	122.9	118.0	113.1	108.3	90.8	85.9	81.0	78.5	1.06
6.1	110.9	104.8	96.6	109.2	104.9	100.5	96.2	80.8	76.4	72.0	69.8	1.34
4.5	99.8	94.3	86.9	98.3	94.4	90.5	86.6	72.7	68.8	64.8	62.8	1.66
5.0	90.7	85.7	79.0	89.4	85.8	82.2	78.7	66.1	62.5	58.9	57.1	2.00
7.1	83.1	78.6	72.4	81.9	78.7	75.4	72.2	60.6	57.3	54.0	52.4	2.38
0.4	76.7	72.5	66.8	75.6	72.6	69.6	66.6	55.9	52.9	49.9	48.3	2.80
74.6	71.3	67.3	62.1	70.2	67.4	64.6	61.9	51.9	49.1	46.3	44.9	3.24
9.6	66.5	62.9	57.9	65.5	62.9	60.3	57.7	48.5	45.8	43.2	41.9	3.72
5.3	62.4	58.9	54.3	61.4	59.0	56.5	54.1	45.4	43.0	40.5	39.3	4.24
1.5	58.7	55.5	51.1	57.8	55.5	53.2	50.9	42.8	40.4	38.1	37.0	4.78
58.0	55.4	52.4	48.3	54.6	52.4	50.3	48.1	40.4	38.2	36.0	34.9	5.36
55.0	52.5	49.6	45.7	51.7	49.7	47.6	45.6	38.3	36.2	34.1	33.1	5.98
52.2	49.9	47.1	43.4	49.2	47.2	45.2	43.3	36.3	34.4	32.4	31.4	6.62
49.7	47.5	44.9	41.4	46.8	44.9	43.1	41.2	34.6	32.7	30.9	29.9	7.30
47.5	45.3	42.9	39.5	44.7	42.9	41.1	39.4	33.0	31.3	29.5	28.6	8.01
45.4	43.4	41.0	37.8	42.7	41.0	39.3	37.7	31.6	29.9	28.2	27.3	8.76
43.5	41.6	39.3	36.2	41.0	39.3	37.7	36.1	30.3	28.6	27.0	26.2	9.53
41.8	39.9	37.7	34.8	39.3	37.8	36.2	34.6	29.1	27.5	25.9	25.1	10.35
40.2	38.4	36.3	33.4	37.8	36.3	34.8	33.3	28.0	26.4	24.9	24.2	11.19
38.7	37.0	34.9	32.2	36.4	35.0	33.5	32.1	26.9	25.5	24.0	23.3	12.07
37.3	35.6	33.7	31.0	35.1	33.7	32.3	30.9	26.0	24.6	23.2	22.4	12.98
36.0	34.4	32.5	30.0	33.9	32.5	31.2	29.9	25.1	23.7	22.4	21.7	13.92
34.8	33.3	31.4	29.0	32.8	31.5	30.2	28.9	24.2	22.9	21.6	20.9	14.90
33.7	32.2	30.4	28.0	31.7	30.4	29.2	27.9	23.4	22.2	20.9	20.3	15.91
32.6	31.2	29.5	27.2	30.7	29.5	28.3	27.1	22.7	21.5	20.3	19.6	16.95
31.7	30.2	28.6	26.3	...	...	...	...	...	...	...	...	18.03
30.7	29.3	27.7	25.6	...	...	...	...	...	...	...	...	19.13
29.8	28.5	26.9	24.8	...	...	...	...	...	...	...	...	20.28
29.0	27.7	26.2	24.1	...	...	...	...	...	...	...	...	21.45
28.2	27.0	25.5	23.5	...	...	...	...	...	...	...	...	22.66
27.5	26.3	24.8	22.9	...	...	...	...	...	...	...	...	23.90

bove the upper heavy lines will cause maximum allowable shears in
e, also, paragraphs in text and foot-note with same, page 567, relating
ckling in beams
elow the lower broken lines will cause *excessive deflections*

ocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table IV* (Continued). Safe Uniform
for Steel I B

Maximum bending stress,

Span, ft	Depth and weight o						
	15-in	12-in					
	36 lb	55 lb	50 lb	45 lb	40 lb	35 lb	31½ lb
..	...	197.0	...	...	...	...	...
3	...	190.2	167.8	138.2	...	164.6	...
4	...	142.7	134.8	127.0	110.4	101.5	84.0
5	...	114.1	107.9	101.6	95.6	81.2	76.7
..	...	...	...	...	...	...	...
6	86.7	95.1	89.9	84.7	79.7	67.6	63.9
7	82.3	81.5	77.0	72.6	68.3	58.0	54.8
8	72.0	71.3	67.4	63.5	59.8	50.7	48.0
9	64.0	63.4	59.9	56.4	53.1	45.1	42.6
10	57.6	57.1	53.9	50.8	47.8	40.6	38.4
11	52.4	51.9	49.0	46.2	43.5	36.9	34.9
12	48.0	47.6	44.9	42.3	39.8	33.8	32.0
13	44.3	43.9	41.5	39.1	36.8	31.2	29.5
14	41.2	40.8	38.5	36.3	34.2	29.0	27.4
15	38.4	38.0	36.0	33.9	31.9	27.1	25.6
16	36.0	35.7	33.7	31.7	29.9	25.4	24.0
17	33.9	33.6	31.7	29.9	28.1	23.9	22.6
18	32.0	31.7	30.0	28.2	26.6	22.5	21.3
19	30.3	30.0	28.4	26.7	25.2	21.4	20.2
20	28.8	28.5	27.0	25.4	23.9	20.3	19.2
21	27.4	27.2	25.7	24.2	22.8	19.3	18.3
22	26.2	25.9	24.5	23.1	21.7	18.4	17.4
23	25.1	24.8	23.4	22.1	20.8	17.6	16.7
24	24.0	23.8	22.5	21.2	19.9	16.9	16.0
25	23.0	22.8	21.6	20.3	19.1	16.2	15.3
26	22.2	21.9	20.7	19.5	18.4	15.6	14.8
27	21.3	...	...	...	...	...	...
28	20.6	...	...	...	...	...	...
29	19.9	...	...	...	...	...	...
30	19.2	...	...	...	...	...	...
31	18.6	...	...	...	...	...	...
32	18.0	...	...	...	...	...	...

Loads above the upper heavy lines will c
webs. See, also, paragraphs in text and foot
to web-buckling in beams.

Loads below the lower broken lines will ca

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* From Pocket Companion, 1915 Edition, Car

Table IV* (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress, 16 000 lb per sq in

an, ft	Depth and weight of sections												Coefficient of deflection
	9-in				8-in					7-in			
	35 lb	30 lb	25 lb	21 lb	25½ lb	23 lb	20½ lb	18 lb	17½ lb	20 lb	17½ lb	15 lb	
...	131.8	102.4	73.4	...	86.6	71.8	57.1	...	...	64.1	49.4	...	......
3	88.3	80.5	72.6	52.2	60.8	57.3	53.9	43.2	...	42.9	39.8	35.0	0.15
4	66.2	60.4	54.5	50.3	45.6	43.0	40.4	37.9	33.6	32.1	29.9	27.6	0.27
5	53.0	48.3	43.6	40.3	36.5	34.4	32.3	30.3	31.1	25.7	23.9	22.1	0.41
6	44.2	40.2	36.3	33.6	30.4	28.7	26.9	25.3	25.9	21.4	19.9	18.4	0.60
7	37.9	34.5	31.1	28.8	26.1	24.6	23.1	21.7	22.2	18.4	17.1	15.8	0.81
8	33.1	30.2	27.2	25.2	22.8	21.5	20.2	19.0	19.4	16.1	14.9	13.8	1.06
9	29.4	26.8	24.2	22.4	20.3	19.1	18.0	16.9	17.3	14.3	13.3	12.3	1.34
10	26.5	24.1	21.8	20.1	18.2	17.2	16.2	15.2	15.6	12.9	11.9	11.0	1.66
11	24.1	22.0	19.8	18.3	16.6	15.6	14.7	13.8	14.1	11.7	10.9	10.0	2.00
12	22.1	20.1	18.2	16.8	15.2	14.3	13.5	12.6	13.0	10.7	10.0	9.2	2.38
13	20.4	18.6	16.8	15.5	14.0	13.2	12.4	11.7	12.0	9.9	9.2	8.5	2.80
14	18.9	17.2	15.6	14.4	13.0	12.3	11.5	10.8	11.1	9.2	8.5	7.9	3.24
15	17.7	16.1	14.5	13.4	12.2	11.5	10.8	10.1	10.4	8.6	8.0	7.4	3.72
16	16.6	15.1	13.6	12.6	11.4	10.8	10.1	9.5	9.7	8.0	7.5	6.9	4.24
17	15.6	14.2	12.8	11.8	10.7	10.1	9.5	8.9	9.2	...	...	...	4.78
18	14.7	13.4	12.1	11.2	10.1	9.6	9.0	8.4	8.6	...	...	...	5.36
19	13.9	12.7	11.5	10.6	...	...	...	...	...	...	...	...	5.98
20	13.3	12.1	10.9	10.1	...	...	...	...	...	...	...	...	6.62

an, ft	Depth and weight of sections													Coefficient of deflection
	6-in			5-in			4-in				3-in			
	17¼ lb	14¾ lb	12¼ lb	14¾ lb	12¼ lb	9¾ lb	10½ lb	9½ lb	8½ lb	7½ lb	7½ lb	6½ lb	5½ lb	
...	...	...	...	...	...	...	...	...	...	...	21.7	...	...	......
1	57.0	...	...	50.4	35.7	...	32.8	27.0	21.0	...	20.7	15.8	10.2	0.02
2	46.6	42.0	27.6	32.3	29.1	21.0	19.0	18.0	16.9	15.7	10.4	9.6	8.8	0.07
3	31.0	28.4	25.8	21.5	19.4	17.2	12.7	12.0	11.3	10.6	6.9	6.4	5.9	0.15
4	23.3	21.3	19.4	16.2	14.5	12.9	9.5	9.0	8.5	8.0	5.2	4.8	4.4	0.27
5	18.6	17.1	15.5	12.9	11.6	10.3	7.6	7.2	6.8	6.4	4.1	3.8	3.5	0.41
6	15.5	14.2	12.9	10.8	9.7	8.6	6.3	6.0	5.6	5.3	3.5	3.2	2.9	0.60
7	13.3	12.2	11.1	9.2	8.3	7.4	5.4	5.1	4.8	4.5	3.0	2.7	2.5	0.81
8	11.6	10.7	9.7	8.1	7.3	6.4	4.8	4.5	4.2	4.0	2.6	2.4	2.2	1.06
9	10.3	9.5	8.6	7.2	6.5	5.7	4.2	4.0	3.8	3.5	..	..	..	1.34
10	9.3	8.5	7.7	6.5	5.8	5.2	3.8	3.6	3.4	3.2	..	..	..	1.66
11	8.5	7.8	7.0	5.9	5.3	4.7	...	..	..	..	..	..	..	2.00
12	7.8	7.1	6.5	5.4	4.8	4.3	...	..	..	..	..	..	..	2.38
13	7.2	6.6	6.0	...	...	...	...	..	..	..	..	..	..	2.80
14	6.7	6.1	5.5	...	...	...	...	..	..	..	..	..	..	3.24

Loads above the upper heavy lines will cause maximum allowable shears in webs. See, also, paragraphs in text and foot-note with same, page 567, relating to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

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* From Pocket *Companion, 1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

Table V.* **Safe Uniform Loads in Units of 1 000 Pounds for S**
Channels

Maximum bending stress, 16 000 lb per sq in

Span, ft	Depth and weight of sections											
	15-in						13-in					
	55 lb	50 lb	45 lb	40 lb	35 lb	33 lb	50 lb	45 lb	40 lb	37 lb	35 lb	32 lb
....	245.4	216.0	186.0	...	...	...	205.7	176.3	...	...	...	...
3	204.0	190.9	177.8	157.2	127.8	120.0	171.6	160.3	146.9	129.2	117.5	97.5
4	153.0	143.2	133.4	123.6	113.8	111.1	128.7	120.2	111.7	106.6	103.2	97.5
5	122.4	114.5	106.7	98.9	91.0	88.9	103.0	96.2	89.4	85.3	82.6	78.0
6	102.0	95.4	88.9	82.4	75.8	74.1	85.8	80.2	74.5	71.1	68.8	65.0
7	87.4	81.8	76.2	70.6	65.0	63.5	73.6	68.7	63.8	60.9	59.0	55.7
8	76.5	71.6	66.7	61.8	56.9	55.6	64.4	60.1	55.9	53.3	51.6	48.7
9	68.0	63.6	59.3	54.9	50.6	49.4	57.2	53.4	49.7	47.4	45.9	43.3
10	61.2	57.3	53.3	49.4	45.5	44.5	51.5	48.1	44.7	42.7	41.3	39.0
11	55.6	52.1	48.5	44.9	41.4	40.4	46.8	43.7	40.6	38.8	37.5	35.4
12	51.0	47.7	44.5	41.2	37.9	37.0	42.9	40.1	37.2	35.5	34.4	32.5
13	47.1	44.1	41.0	38.0	35.0	34.2	39.6	37.0	34.4	32.8	31.8	30.0
14	43.7	40.9	38.1	35.3	32.5	31.8	35.8	34.4	31.9	30.5	29.5	27.9
15	40.8	38.2	35.6	33.0	30.3	29.6	34.3	32.1	29.8	28.4	27.5	26.0
16	38.2	35.8	33.3	30.9	28.4	27.8	32.2	30.1	27.9	26.7	25.8	24.4
17	36.0	33.7	31.4	29.1	26.8	26.1	30.3	28.3	26.3	25.1	24.3	22.9
18	34.0	31.8	29.6	27.5	25.3	24.7	28.6	26.7	24.8	23.7	22.9	21.7
19	32.2	30.1	28.1	26.0	23.9	23.4	27.1	25.3	23.5	22.4	21.7	20.5
20	30.6	28.6	26.7	24.7	22.8	22.3	25.7	24.0	22.3	21.3	20.6	19.5
21	29.1	27.3	25.4	23.5	21.7	21.2	24.5	22.9	21.3	20.3	19.7	18.6
22	27.8	26.0	24.3	22.5	20.7	20.2	23.4	21.9	20.3	19.4	18.8	17.7
23	26.6	24.9	23.2	21.5	19.8	19.3	22.4	20.9	19.4	18.5	18.0	17.0
24	25.5	23.9	22.2	20.6	19.0	18.5	21.5	20.0	18.6	17.8	17.2	16.2
25	24.5	22.9	21.3	19.8	18.2	17.8	20.6	19.2	17.9	17.1	16.5	15.6
26	23.5	22.0	20.5	19.0	17.5	17.1	19.8	18.5	17.2	16.4	15.9	15.0
27	22.7	21.2	19.8	18.3	16.9	16.5	19.1	17.8	16.6	15.8	15.3	14.4
28	21.9	20.5	19.1	17.7	16.3	15.9	18.4	17.2	16.0	15.2	14.7	13.9
29	21.1	19.7	18.4	17.0	15.7	15.3	...	...	...	...	...	...
30	20.4	19.1	17.8	16.5	15.2	14.8	...	...	...	...	...	...
31	19.7	18.5	17.2	15.9	14.7	14.3	...	...	...	...	...	...
32	19.1	17.9	16.7	15.4	14.2	13.9	...	...	...	...	...	...

Loads above the upper heavy lines will cause maximum allowable s webs. See, also, paragraphs in text and foot-note with same, page 567. to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

ble V* (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel Channels

Maximum bending stress, 16 000 lb per sq in

n.	Depth and weight of sections										Coefficient of deflection
	12-in					10-in					
	40 lb	35 lb	30 lb	25 lb	20½ lb	35 lb	30 lb	25 lb	20 lb	15 lb	
	181.9	....	....	....	....	164.6	135.2	105.8	....	....	....
	175.1	152.6	123.1	93.6	....	123.2	110.1	97.0	76.4	48.0	0.07
	116.7	106.2	95.8	85.3	67.2	82.1	73.4	64.7	56.0	47.6	0.15
	87.5	79.7	71.8	64.0	56.9	61.6	55.1	48.5	42.0	35.7	0.27
	70.0	63.7	57.5	51.2	45.5	49.3	44.0	38.8	33.6	28.5	0.41
	58.4	53.1	47.9	42.7	38.0	41.1	36.7	32.3	28.0	23.8	0.60
	50.0	45.5	41.1	36.6	32.5	35.2	31.5	27.7	24.0	20.4	0.81
	43.8	39.8	35.9	32.0	28.5	30.8	27.5	24.3	21.0	17.8	1.06
	38.9	35.4	31.9	28.4	25.3	27.4	24.5	21.6	18.7	15.9	1.34
	35.0	31.9	28.7	25.6	22.8	24.6	22.0	19.4	16.8	14.3	1.66
	31.8	29.0	26.1	23.3	20.7	22.4	20.0	17.6	15.3	13.0	2.00
	29.2	26.6	23.9	21.3	19.0	20.5	18.4	16.2	14.0	11.9	2.38
	26.9	24.5	22.1	19.7	17.5	19.0	16.9	14.9	12.9	11.0	2.80
	25.0	22.8	20.5	18.3	16.3	17.6	15.7	13.9	12.0	10.2	3.24
	23.3	21.2	19.2	17.1	15.2	16.4	14.7	12.9	11.2	9.5	3.72
	21.9	19.9	18.0	16.0	14.2	15.4	13.8	12.1	10.5	8.9	4.24
	20.6	18.7	16.9	15.1	13.4	14.5	13.0	11.4	9.9	8.4	4.78
	19.5	17.7	16.0	14.2	12.7	13.7	12.2	10.8	9.3	7.9	5.36
	18.4	16.8	15.1	13.5	12.0	13.0	11.6	10.2	8.8	7.5	5.98
	17.5	15.9	14.4	12.8	11.4	12.3	11.0	9.7	8.4	7.1	6.62
	16.7	15.2	13.7	12.2	10.8	11.7	10.5	9.2	8.0	6.8	7.30
	15.9	14.5	13.1	11.6	10.4	11.2	10.0	8.8	7.6	6.5	8.01
	15.2	13.9	12.5	11.1	9.9	....	....	....	....	....	8.76
	14.6	13.3	12.0	10.7	9.5	....	....	....	....	....	9.53
	14.0	12.8	11.5	10.2	9.1	....	....	....	....	....	10.35
	13.5	12.3	11.1	9.8	8.8	....	....	....	....	....	11.19

oads above the upper heavy lines will cause maximum allowable shears in bs. See, also, paragraphs in text and foot-note with same, page 567, relating web-buckling in beams.

oads below the lower broken lines will cause excessive deflections

rom Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table V* (Continued). Safe Uniform Loads in Units of 1 000 Po

Steel Channels

Maximum bending stress, 16 000 lb per sq in

Span, ft	Depth and weight of sections												
	9-in				8-in					7-in			
	25 lb	20 lb	15 lb	13¼ lb	21¼ lb	18¾ lb	16¼ lb	13¾ lb	11¼ lb	19¾ lb	17¼ lb	14¾ lb	12¼ lb
....	110.7	81.4	..	..	93.1	78.4	63.8	49.1	..	88.6	73.9	59.2	44.5
2	83.8	72.0	51.8	41.4	63.7	58.5	53.2	48.0	35.2	50.6	46.0	41.4	36.8
3	55.9	48.0	40.2	37.4	42.5	39.0	35.5	32.0	28.7	33.7	30.7	27.6	24.6
4	41.9	36.0	30.1	28.0	31.8	29.2	26.6	24.0	21.5	25.3	23.0	20.7	18.4
5	33.5	28.8	24.1	22.4	25.5	23.4	21.3	19.2	17.2	20.2	18.4	26.6	14.7
6	27.9	24.0	20.1	18.7	21.2	19.5	17.7	16.0	14.4	16.9	15.3	13.8	12.3
7	23.9	20.6	17.2	16.0	18.2	16.7	15.2	13.7	12.3	14.4	13.1	11.8	10.5
8	20.9	18.0	15.1	14.0	15.9	14.6	13.3	12.0	10.8	12.6	11.5	10.4	9.2
9	18.6	16.0	13.4	12.5	14.2	13.0	11.8	10.7	9.6	11.2	10.2	9.2	8.2
10	16.8	14.4	12.1	11.2	12.7	11.7	10.6	9.6	8.6	10.1	9.2	8.3	7.4
11	15.2	13.1	11.0	10.2	11.6	10.6	9.7	8.7	7.8	9.2	8.4	7.5	6.7
12	14.0	12.0	10.1	9.3	10.6	9.7	8.9	8.0	7.2	8.4	7.7	6.9	6.1
13	12.9	11.1	9.3	8.6	9.8	9.0	8.2	7.4	6.6	7.8	7.1	6.4	5.7
14	12.0	10.3	8.6	8.0	9.1	8.4	7.6	6.9	6.2	7.2	6.6	5.9	5.3
15	11.2	9.6	8.0	7.5	8.5	7.8	7.1	6.4	5.7	6.7	6.1	5.5	4.9
16	10.5	9.0	7.5	7.0	8.0	7.3	6.7	6.0	5.4	6.3	5.7	5.2	4.6
17	9.9	8.5	7.1	6.6	7.5	6.9	6.3	5.6	5.1	..	..	..	..
18	9.3	8.0	6.7	6.2	7.1	6.5	5.9	5.3	4.8	..	..	..	..
19	8.8	7.6	6.3	5.9	..	..	..	..	..	..	..	..	..
20	8.4	7.2	6.0	5.6	..	..	..	..	..	..	..	..	..

Span, ft	Depth and weight of sections											
	6-in				5-in			4-in			3-in	
	15½ lb	13 lb	10½ lb	8 lb	11½ lb	9 lb	6½ lb	7¼ lb	6¼ lb	5¼ lb	6 lb	5 lb
....	...	...	...	...	47.7	...	..	26.0	..	..	21.7	15.8
1	67.6	52.8	38.2	24.0	44.4	33.0	19.0	24.4	20.2	14.4	14.7	13.1
2	34.7	30.8	26.9	23.1	22.2	18.9	15.8	12.2	11.1	10.1	7.4	6.6
3	23.2	20.5	17.9	15.4	14.8	12.6	10.5	8.1	7.4	6.7	4.9	4.4
4	17.4	15.4	13.4	11.6	11.1	9.5	7.9	6.1	5.6	5.1	3.7	3.3
5	13.9	12.3	10.8	9.2	8.9	7.6	6.3	4.9	4.5	4.1	2.9	2.6
6	11.6	10.3	9.0	7.7	7.4	6.3	5.3	4.1	3.7	3.4	2.5	2.2
7	9.9	8.8	7.7	6.6	6.3	5.4	4.5	3.5	3.2	2.9	2.1	1.9
8	8.7	7.7	6.7	5.8	5.5	4.7	4.0	3.0	2.8	2.5	1.8	1.6
9	7.7	6.8	6.0	5.1	4.9	4.2	3.5	2.7	2.5	2.2	..	..
10	6.9	6.2	5.4	4.6	4.4	3.8	3.2	2.4	2.2	2.0	..	..
11	6.3	5.6	4.9	4.2	4.0	3.4	2.9	..	..	..	..	..
12	5.8	5.1	4.5	3.9	3.7	3.2	2.6	..	..	..	..	..
13	5.3	4.7	4.1	3.6	...	...	..	..	..	..	..	..
14	5.0	4.4	3.8	3.3	...	...	..	..	..	..	..	..

Loads above the upper heavy lines will cause maximum allowable s webs. See, also, paragraphs in text and foot-note with same, page 567, to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsb

Table VI.* Safe Uniform Loads in Units of 1 000 Pounds for Steel H Beams

Maximum bending stress, 16 000 lb per sq in

Span, ft	Depth and weight of sections				Coefficients of deflection
	8-in 34.0-lb	6-in 23.8-lb	5-in 18.7-lb	4-in 13.6-lb	
......	....	....	....	25.0	....
3	....	....	31.3	19.0	0.15
4	....	37.6	25.4	14.3	0.27
5	....	32.1	20.3	11.4	0.41
......	60.0	....	....	....	....
6	51.3	26.7	16.9	9.5	0.60
7	44.0	22.9	14.5	8.1	0.81
8	38.5	20.1	12.7	7.1	1.06
9	34.2	17.8	11.3	6.3	1.34
10	30.8	16.0	10.1	5.7	1.66
11	28.0	14.6	9.2	....	2.00
12	25.6	13.4	8.5	....	2.38
13	23.7	12.3	....	....	2.80
14	22.0	11.5	....	....	3.24
15	20.5	....	....	....	3.72
16	19.2	....	....	....	4.24
17	18.1	....	....	....	4.78
18	17.1	....	....	....	5.36

Table VII.† Safe Uniform Loads in Pounds for Small Steel Channels, or Grooved Steel

Computed for a fiber-stress of 16 000 lb per sq in

For dimensions of sections, see Table IX, page 360

Section-number	Depth, in	Weight per foot, lb	Span in feet							
			2	2.5	3	3.5	4	4.5	5	6
1	2¼	3.80	3 785	3 028	2 523	2 163	1 892	1 682	1 514	1 261
2	2	2.90	2 560	2 048	1 706	1 463	1 280	1 138	1 024	853
3	2	3.60	2 880	2 304	1 920	1 643	1 440	1 280	1 152	960
4	2	3.60	3 120	2 496	2 080	1 783	1 560	1 386	1 248	1 040
5	2	2.60	2 256	1 804	1 504	1 289	1 128	1 000	902	752
6	2	2.00	1 418	1 134	945	810	709	630	567	472
7	1¾	1.13	907	726	605	518	454	403	363	302
8	1½	1.32	768	614	512	439	384	341	307	256
9	1½	1.46	868	694	578	496	434	386	347	289
10	1¼	0.94	475	380	316	271	237	211	190	....
11	1⅛	1.12	469	375	313	268	234	208	188	....
12	1⅛	1.00	437	350	291	250	218	194	175	....
13	1	0.83	336	268	224	192	168	....	....	....
14	1	0.68	266	212	177	152	133	....	....	....
15	⅞	0.67	224	180	149	128	112	....	....	....
16	⅞	0.69	229	183	152	130	....	....	....	....
17	¾	0.53	133	106	88	.....	....	....	....	....

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* From *Pocket Companion, 1915 Edition, Carnegie Steel* Company, Pittsburgh, Pa.

† *Compiled by F. E. Kidder. See note on page 570.*

Table VIII.* **Safe Uniform Loads in Units of 1 000 Pounds for**
Angles with Equal Legs
Neutral Axis Parallel to Either Leg
Maximum bending stress, 16 000 lb per sq in

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft
8×8	1⅛	186.99	8.31	22.5
8×8	1^1/16	177.81	7.87	22.6
8×8	1	168.53	7.43	22.7
8×8	15/16	159.15	6.98	22.8
8×8	⅞	149.55	6.53	22.9
8×8	13/16	139.84	6.08	23.0
8×8	¾	130.03	5.63	23.1
8×8	11/16	120.00	5.18	23.2
8×8	⅝	109.87	4.73	23.2
8×8	9/16	99.63	4.28	23.3
8×8	½	89.28	3.82	23.4
6×6	1	91.41	5.48	16.7
6×6	15/16	86.51	5.16	16.8
6×6	⅞	81.39	4.84	16.8
6×6	13/16	76.27	4.51	16.9
6×6	¾	71.04	4.18	17.0
6×6	11/16	65.81	3.85	17.1
6×6	⅝	60.37	3.51	17.2
6×6	9/16	54.83	3.17	17.3
6×6	½	49.17	2.83	17.4
6×6	7/16	43.41	2.48	17.5
6×6	⅜	37.65	2.14	17.6
5×5	1	61.87	4.55	13.6
5×5	15/16	58.56	4.28	13.7
5×5	⅞	55.15	4.00	13.8
5×5	13/16	51.73	3.73	13.9
5×5	¾	48.32	3.45	14.0
5×5	11/16	44.80	3.18	14.1
5×5	⅝	41.17	2.90	14.2
5×5	9/16	37.44	2.62	14.3
5×5	½	33.60	2.34	14.4
5×5	7/16	29.76	2.06	14.5
5×5	⅜	25.81	1.78	14.5
4×4	13/16	32.11	2.95	10.9
4×4	¾	29.97	2.73	11.0
4×4	11/16	27.84	2.51	11.1
4×4	⅝	25.60	2.29	11.2
4×4	9/16	23.36	2.07	11.3
4×4	½	21.01	1.85	11.4
4×4	7/16	18.67	1.63	11.4
4×4	⅜	16.21	1.41	11.5
4×4	5/16	13.76	1.19	11.6
4×4	¼	11.20	0.96	11.7

Size, in	Thickness, in	1-ft span
		Safe load
3½×3½	13/16	24.00
3½×3½	¾	22.51
3½×3½	11/16	20.91
3½×3½	⅝	19.31
3½×3½	9/16	17.60
3½×3½	½	15.89
3½×3½	7/16	14.08
3½×3½	⅜	12.27
3½×3½	5/16	10.45
3½×3½	¼	8.43
3 ×3	⅝	13.87
3 ×3	9/16	12.69
3 ×3	½	11.41
3 ×3	7/16	10.13
3 ×3	⅜	8.85
3 ×3	5/16	7.57
3 ×3	¼	6.19
2½×2½	½	7.79
2½×2½	7/16	6.93
2½×2½	⅜	6.08
2½×2½	5/16	5.12
2½×2½	¼	4.16
2½×2½	3/16	3.20
2½×2½	⅛	2.13
2 ×2	7/16	4.27
2 ×2	⅜	3.73
2 ×2	5/16	3.20
2 ×2	¼	2.67
2 ×2	3/16	2.03
2 ×2	⅛	1.39
1¾×1¾	7/16	3.20
1¾×1¾	⅜	2.77
1¾×1¾	5/16	2.45
1¾×1¾	¼	2.03
1¾×1¾	3/16	1.49
1¾×1¾	⅛	1.07
1½×1½	⅜	2.03
1½×1½	5/16	1.71
1½×1½	¼	1.39
1½×1½	3/16	1.07
1½×1½	⅛	0.77
1¼×1¼	5/16	1.17
1¼×1¼	¼	0.97
1¼×1¼	3/16	0.76
1¼×1¼	⅛	0.52
1 ×1	¼	0.60
1 ×1	3/16	0.47
1 ×1	⅛	0.33

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitt

ble IX.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Angles with Unequal Legs

Neutral Axis Parallel to Shorter Leg

Maximum bending stress, 16 000 lb per sq in

Thickness, in	1-ft span	Maximum span, 360 × deflection	
	Safe load	Safe load	Length, ft
1	161.17	7.49	21.5
15/16	152.21	7.04	21.6
7/8	143.04	6.59	21.7
13/16	133.87	6.14	21.8
3/4	124.48	5.68	21.9
11/16	114.88	5.22	22.0
5/8	105.28	4.76	22.1
9/16	95.47	4.30	22.2
1/2	85.55	3.84	22.3
7/16	75.41	3.37	22.4
1	146.03	7.53	19.4
15/16	138.03	7.08	19.5
7/8	129.92	6.63	19.6
13/16	121.60	6.17	19.7
3/4	113.17	5.72	19.8
11/16	104.58	5.23	19.9
5/8	95.79	4.78	20.0
9/16	86.93	4.32	20.1
1/2	77.97	3.86	20.2
7/16	68.80	3.39	20.3
1	112.85	6.52	17.3
15/16	106.67	6.13	17.4
7/8	100.48	5.75	17.5
13/16	94.08	5.36	17.6
3/4	87.68	4.97	17.6
11/16	81.07	4.58	17.7
5/8	74.35	4.18	17.8
9/16	67.52	3.77	17.9
1/2	60.59	3.37	18.0
7/16	53.44	2.96	18.1
3/8	46.19	2.54	18.2
1	85.55	5.56	15.4
15/16	80.96	5.22	15.5
7/8	76.27	4.89	15.6
13/16	71.47	4.55	15.7
3/4	66.67	4.22	15.8
11/16	61.65	3.88	15.9
5/8	56.64	3.54	16.0
9/16	51.52	3.20	16.1
1/2	46.19	2.85	16.2
7/16	40.85	2.51	16.3
3/8	35.41	2.16	16.4

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft
6×3½	1	83.52	5.57	15.0
6×3½	15/16	79.04	5.24	15.1
6×3½	7/8	74.45	4.90	15.2
6×3½	13/16	69.87	4.57	15.3
6×3½	3/4	65.07	4.23	15.4
6×3½	11/16	60.27	3.89	15.5
6×3½	5/8	55.36	3.55	15.6
6×3½	9/16	50.35	3.21	15.7
6×3½	1/2	45.23	2.86	15.8
6×3½	7/16	40.00	2.52	15.9
6×3½	3/8	34.67	2.17	16.0
6×3½	5/16	29.23	1.83	16.0
5×4	7/8	53.23	4.00	13.3
5×4	13/16	50.03	3.73	13.4
5×4	3/4	46.61	3.46	13.5
5×4	11/16	43.20	3.19	13.5
5×4	5/8	39.79	2.92	13.6
5×4	9/16	36.16	2.64	13.7
5×4	1/2	32.53	2.36	13.8
5×4	7/16	28.80	2.07	13.9
5×4	3/8	24.96	1.78	14.0
5×3½	7/8	52.05	4.04	12.9
5×3½	13/16	48.85	3.76	13.0
5×3½	3/4	45.65	3.49	13.1
5×3½	11/16	42.35	3.21	13.2
5×3½	5/8	38.93	2.93	13.3
5×3½	9/16	35.41	2.64	13.4
5×3½	1/2	31.89	2.36	13.5
5×3½	7/16	28.16	2.07	13.6
5×3½	3/8	24.43	1.79	13.7
5×3½	5/16	20.69	1.51	13.7
5×3	13/16	47.47	3.77	12.6
5×3	3/4	44.37	3.49	12.7
5×3	11/16	41.17	3.22	12.8
5×3	5/8	37.87	2.94	12.9
5×3	9/16	34.45	2.65	13.0
5×3	1/2	31.04	2.37	13.1
5×3	7/16	27.52	2.09	13.2
5×3	3/8	23.89	1.80	13.3
5×3	5/16	20.16	1.51	13.4

n Pocket *Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.*

Table IX* (Continued). Safe Uniform Loads in Units of 1 000
for Steel Angles with Unequal Legs
Neutral Axis Parallel to Shorter Leg
Maximum bending stress, 16 000 lb per sq in

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection		Size, in	Thickness, in	1-ft span
		Safe load	Safe load	Length, ft			Safe load
4½×3	13/16	38.61	3.36	11.5	3 ×2½	9/16	12.27
4½×3	¾	36.05	3.11	11.6	3 ×2½	½	11.09
4½×3	11/16	33.49	2.87	11.7	3 ×2½	7/16	9.92
4½×3	⅝	30.83	2.62	11.8	3 ×2½	⅜	8.64
4½×3	9/16	28.16	2.38	11.8	3 ×2½	5/16	7.36
4½×3	½	25.28	2.13	11.9	3 ×2½	¼	5.97
4½×3	7/16	22.40	1.87	12.0			
4½×3	⅜	19.52	1.61	12.1	3 ×2	½	10.67
4½×3	5/16	16.43	1.35	12.2	3 ×2	7/16	9.49
4 ×3½	13/16	31.15	2.94	10.6	3 ×2	⅜	8.32
4 ×3½	¾	29.23	2.73	10.7	3 ×2	5/16	7.04
4 ×3½	11/16	27.20	2.52	10.8	3 ×2	¼	5.76
4 ×3½	⅝	25.07	2.30	10.9			
4 ×3½	9/16	22.93	2.08	11.0	2½×2	½	7.47
4 ×3½	½	20.69	1.86	11.1	2½×2	7/16	6.72
4 ×3½	7/16	18.35	1.64	11.2	2½×2	⅜	5.87
4 ×3½	⅜	16.00	1.41	11.3	2½×2	5/16	5.01
4 ×3½	5/16	13.44	1.18	11.4	2½×2	¼	4.05
4 ×3	13/16	30.61	2.97	10.3	2½×2	3/16	3.09
4 ×3	¾	28.59	2.75	10.4	2½×2	⅛	2.13
4 ×3	11/16	26.56	2.53	10.5			
4 ×3	⅝	24.53	2.31	10.6	2½×1½	5/16	4.69
4 ×3	9/16	22.40	2.09	10.7	2½×1½	¼	3.84
4 ×3	½	20.16	1.87	10.8	2½×1½	3/16	2.99
4 ×3	7/16	17.92	1.64	10.9			
4 ×3	⅜	15.57	1.42	11.0	2¼×1½	½	5.76
4 ×3	5/16	13.12	1.19	11.0	2¼×1½	7/16	5.12
4 ×3	¼	10.67	0.96	11.1	2¼×1½	⅜	4.48
3½×3	13/16	23.47	2.57	9.1	2¼×1½	5/16	3.86
3½×3	¾	21.87	2.38	9.2	2¼×1½	¼	3.20
3½×3	11/16	20.37	2.19	9.3	2¼×1½	3/16	2.45
3½×3	⅝	18.77	2.00	9.4			
3½×3	9/16	17.17	1.81	9.5	2 ×1½	⅜	3.63
3½×3	½	15.47	1.62	9.5	2 ×1½	5/16	3.09
3½×3	7/16	13.76	1.43	9.6	2 ×1½	¼	2.56
3½×3	⅜	12.05	1.24	9.7	2 ×1½	3/16	1.92
3½×3	5/16	10.24	1.05	9.8	2 ×1½	⅛	1.39
3½×3	¼	8.32	0.84	9.9	2 ×1¼	¼	2.45
3½×2½	11/16	19.73	2.19	9.0	2 ×1¼	3/16	1.92
3½×2½	⅝	18.24	2.00	9.1			
3½×2½	9/16	16.64	1.82	9.1	1¾×1¼	¼	1.92
3½×2½	½	15.04	1.63	9.2	1¾×1¼	3/16	1.49
3½×2½	7/16	13.44	1.44	9.3	1¾×1¼	⅛	1.00
3½×2½	⅜	11.73	1.24	9.4	1½×1¼	5/16	1.71
3½×2½	5/16	9.92	1.04	9.5	1½×1¼	¼	1.39
3½×2½	¼	8.00	0.83	9.6	1½×1¼	3/16	1.07

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitt

rm Loads in Units of 1 000 Pounds for Steel Angles with Unequal Legs

utral Axis Parallel to Longer Leg

mum bending stress, 16 000 lb per sq in

Maximum span, 360 × deflection		Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
Safe load	Length, ft			Safe load	Safe load	Length, ft
5.44	17.5	6×3½	1	30.93	3.09	10.0
5.11	17.6	6×3½	15/16	29.23	2.90	10.1
4.79	17.7	6×3½	7/8	27.63	2.71	10.2
4.45	17.8	6×3½	13/16	25.92	2.52	10.3
4.13	17.9	6×3½	3/4	24.21	2.33	10.4
3.80	18.0	6×3½	11/16	22.51	2.14	10.5
3.48	18.0	6×3½	5/8	20.69	1.95	10.6
3.15	18.1	6×3½	9/16	18.88	1.76	10.7
2.81	18.2	6×3½	1/2	16.96	1.57	10.8
2.47	18.3	6×3½	7/16	15.04	1.38	10.9
		6×3½	3/8	13.12	1.19	11.0
3.10	10.4	6×3½	5/16	11.09	1.00	11.1
2.90	10.5					
2.71	10.6					
2.52	10.7	5×4	7/8	35.31	3.15	11.2
2.33	10.8	5×4	13/16	33.17	2.93	11.3
2.13	10.9	5×4	3/4	30.93	2.71	11.4
1.94	11.0	5×4	11/16	28.69	2.50	11.5
1.74	11.1	5×4	5/8	26.45	2.28	11.6
1.57	11.2	5×4	9/16	24.11	2.16	11.7
1.38	11.3	5×4	1/2	21.76	1.84	11.8
		5×4	7/16	19.31	1.62	11.9
3.10	10.2	5×4	3/8	16.75	1.40	12.0
2.90	10.3					
2.71	10.4	5×3½	7/8	26.88	2.71	9.9
2.52	10.5	5×3½	13/16	25.28	2.53	10.0
2.33	10.6	5×3½	3/4	23.68	2.34	10.1
2.14	10.7	5×3½	11/16	21.97	2.15	10.2
1.95	10.8	5×3½	5/8	20.27	1.97	10.3
1.76	10.9	5×3½	9/16	18.45	1.78	10.4
1.57	11.0	5×3½	1/2	16.64	1.60	10.4
1.38	11.1	5×3½	7/16	14.83	1.41	10.5
1.19	11.2	5×3½	3/8	12.91	1.22	10.6
		5×3½	5/16	10.88	1.02	10.7
3.55	11.4					
3.33	11.5					
3.12	11.6	5×3	13/16	18.56	2.16	8.6
2.90	11.7	5×3	3/4	17.39	2.00	8.7
2.69	11.8	5×3	11/16	16.11	1.83	8.8
2.47	11.9	5×3	5/8	14.83	1.67	8.9
2.26	12.0	5×3	9/16	13.55	1.51	9.0
2.05	12.0	5×3	1/2	12.27	1.35	9.1
1.84	12.1	5×3	7/16	10.88	1.18	9.2
1.62	12.2	5×3	3/8	9.49	1.02	9.3
1.39	12.3	5×3	5/16	8.00	0.85	9.4

on, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

Table X* (Continued). Safe Uniform Loads in Units of 1 000

Steel Angles with Unequal Legs

Neutral Axis Parallel to Longer Leg

Maximum bending stress, 16 000 lb per sq in

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft
4½×3	13/16	18.24	2.15	8.5
4½×3	¾	17.07	1.99	8.6
4½×3	11/16	15.89	1.83	8.7
4½×3	⅝	14.61	1.67	8.8
4½×3	9/16	13.33	1.51	8.8
4½×3	½	12.05	1.35	8.9
4½×3	7/16	10.77	1.19	9.0
4½×3	⅜	9.39	1.03	9.1
4½×3	5/16	8.00	0.87	9.2
4 ×3½	13/16	24.53	2.56	9.6
4 ×3½	¾	22.93	2.37	9.7
4 ×3½	11/16	21.33	2.18	9.8
4 ×3½	⅝	19.63	1.98	9.9
4 ×3½	9/16	17.92	1.79	10.0
4 ×3½	½	16.21	1.60	10.1
4 ×3½	7/16	14.40	1.41	10.2
4 ×3½	⅜	12.59	1.22	10.3
4 ×3½	5/16	10.67	1.03	10.4
4 ×3	13/16	17.92	2.15	8.3
4 ×3	¾	16.75	1.99	8.4
4 ×3	11/16	15.57	1.83	8.5
4 ×3	⅝	14.40	1.67	8.6
4 ×3	9/16	13.12	1.51	8.7
4 ×3	½	11.84	1.35	8.8
4 ×3	7/16	10.56	1.19	8.9
4 ×3	⅜	9.28	1.03	8.9
4 ×3	5/16	7.89	0.87	9.0
4 ×3	¼	6.40	0.70	9.1
3½×3	13/16	17.60	2.17	8.1
3½×3	¾	16.43	2.01	8.2
3½×3	11/16	15.36	1.85	8.3
3½×3	⅝	14.19	1.69	8.4
3½×3	9/16	12.91	1.52	8.5
3½×3	½	11.73	1.36	8.6
3½×3	7/16	10.45	1.20	8.7
3½×3	⅜	9.07	1.04	8.7
3½×3	5/16	7.68	0.87	8.8
3½×3	¼	6.19	0.70	8.9
3½×2½	11/16	10.56	1.51	7.0
3½×2½	⅝	9.81	1.39	7.1
3½×2½	9/16	8.96	1.26	7.1
3½×2½	½	8.11	1.13	7.2
3½×2½	7/16	7.25	0.99	7.3
3½×2½	⅜	6.29	0.85	7.4
3½×2½	5/16	5.33	0.71	7.5
3½×2½	¼	4.37	0.58	7.6

Size, in	Thickness, in	1-ft span
		Safe load
3 ×2½	9/16	8.75
3 ×2½	½	7.89
3 ×2½	7/16	7.04
3 ×2½	⅜	6.19
3 ×2½	5/16	5.23
3 ×2½	¼	4.27
3 ×2	½	5.01
3 ×2	7/16	4.48
3 ×2	⅜	3.95
3 ×2	5/16	3.41
3 ×2	¼	2.77
2½×2	½	4.91
2½×2	7/16	4.37
2½×2	⅜	3.84
2½×2	5/16	3.31
2½×2	¼	2.67
2½×2	3/16	2.13
2½×2	⅛	1.49
2½×1½	5/16	1.81
2½×1½	¼	1.49
2½×1½	3/16	1.17
2¼×1½	½	2.77
2¼×1½	7/16	2.45
2¼×1½	⅜	2.13
2¼×1½	5/16	1.81
2¼×1½	¼	1.49
2¼×1½	3/16	1.17
2 ×1½	⅜	2.13
2 ×1½	5/16	1.81
2 ×1½	¼	1.49
2 ×1½	3/16	1.17
2 ×1½	⅛	0.80
2 ×1¼	¼	1.04
2 ×1¼	3/16	0.80
1¾×1¼	¼	1.01
1¾×1¼	3/16	0.80
1¾×1¼	⅛	0.56
1½×1¼	5/16	1.17
1½×1¼	¼	0.99
1½×1¼	3/16	0.78

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company,

I.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Tees with Neutral Axis Parallel to Flange

Maximum bending stress, 16 000 lb per sq in

EQUAL FLANGE AND STEM

em n	Weight per foot, lb	1-ft span	Maximum span, 360 × deflection		Size		Weight per foot, lb	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	L'gth ft	Flnge in	Stem in		Safe load	Safe load	L'gth ft
	13.5	21.55	1.89	11.4	2¼	2¼	4.1	3.41	0.53	6.4
	10.5	16.85	1.45	11.6	2	2	4.3	3.31	0.59	5.6
½	11.7	16.32	1.65	9.9	2	2	3.56	2.77	0.49	5.7
½	9.2	12.69	1.27	10.0	1¾	1¾	3.09	2.03	0.41	4.9
	9.9	11.73	1.41	8.3	1½	1½	2.47	1.49	0.36	4.1
	8.9	10.45	1.24	8.4	1½	1½	1.94	1.17	0.27	4.3
	7.8	9.17	1.08	8.5	1¼	1¼	2.02	1.01	0.30	3.4
	6.7	7.89	0.92	8.6	1¼	1¼	1.59	0.78	0.22	3.5
½	6.4	6.29	0.90	7.0	1	1	1.25	0.49	0.18	2.7
½	5.5	5.33	0.75	7.1	1	1	0.89	0.35	0.12	2.9
¼	4.9	4.37	0.69	6.3	....	....	......	....	...	....

UNEQUAL FLANGE AND STEM

em n	Weight per foot, lb	1-ft span	Maximum span, 360 × deflection		Size		Weight per foot, lb	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	L'gth ft	Flnge in	Stem in		Safe load	Safe load	L'gth ft
	13.4	11.41	1.25	9.1	3½	3	10.8	12.05	1.42	8.5
½	10.9	8.96	1.20	7.5	3½	3	8.5	9.49	1.09	8.7
½	15.7	22.72	2.37	9.6	3½	3	7.5	9.07	1.04	8.7
	9.8	9.71	1.07	9.1	3	4	11.7	20.69	1.92	10.8
	8.4	8.32	0.90	9.2	3	4	10.5	18.35	1.68	10.9
½	9.2	6.72	0.87	7.7	3	4	9.2	16.11	1.47	11.0
½	7.8	5.76	0.74	7.8	3	3½	10.8	15.89	1.66	9.6
	15.3	33.39	2.40	13.9	3	3½	9.7	14.19	1.46	9.7
	11.9	25.92	1.84	14.1	3	3½	8.5	12.37	1.26	9.8
½	14.4	27.09	2.15	12.6	3	2½	7.1	6.40	0.89	7.2
½	11.2	21.12	1.65	12.8	3	2½	6.1	5.55	0.76	7.3
3	9.2	9.60	1.08	8.9	2½	3	7.1	8.96	1.08	8.3
3	7.8	8.21	0.90	9.1	2½	3	6.1	7.68	0.91	8.4
2½	8.5	6.61	0.87	7.6	2½	1¼	2.87	0.93	0.25	3.7
2½	7.2	5.65	0.73	7.7	2	1½	3.09	1.60	0.36	4.4
2	7.8	4.27	0.70	6.1	1½	2	2.45	2.03	0.37	5.5
2	6.7	3.63	0.59	6.2	1½	1¼	1.25	0.57	0.15	3.7
4	12.6	21.12	1.90	11.1	1¼	⅝	0.88	0.14	0.07	1.9
4	9.8	16.53	1.46	11.3	....	....	......	....	...	....

Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

less than the standard beam, or a saving of 10% in weight
parisons with other sizes of the standard beams previously rol
of this country show that the Bethlehem I beams afford an
capacity, but with practically 10% less weight of metal.

Thickness of Webs and Flanges. It is claimed that the w
beams are much thicker than required for a scientifially prop
It is impossible to reduce the WEB-THICKNESS in the ordinary
the Grey Mill webs of the desired thickness can be produced.
the FLANGES part of the metal thus saved, the strength of the
tained, thereby affording a lighter section of the same stren
FLANGES give increased lateral stiffness, which commends the us
in many cases, where the NARROW FLANGES and lack of sufficien
prevent the use of ordinary standard beams.

Depth and Weight of Bethlehem Beams. Formerly the
rolled in this country were 24 in deep, weighed 115 lb per ft, an
modulus of 246.3. Whenever greater strength was required,
was necessary. Bethlehem beams are rolled to a maximum
weigh 200 lb per ft, and have a section-modulus of 610, or tw
times the strength of the largest beam previously rolled. The
using ROLLED BEAMS instead of BUILT-UP RIVETED GIRDERS is, th
increased. These rolled beams and girders afford a saving in we
and also a large economy in COST OF FABRICATION, as they do
punching, assembling and riveting necessary for building a riv

Bethlehem Girder Beams.[*] BETHLEHEM GIRDER BEAMS, f
in depth, inclusive, have a strength, or section-modulus, equal
minimum-weight STANDARD I BEAMS of the same depth. Th
however, weighs generally 12½% less than the combined wei
standard beams, not considering the saving in weight of separa
assembling the standard beams into a girder. For example, a B
girder beam, weighing 73 lb per ft has a section-modulus of 11
page 358). Two standard 15-in I beams, each weighing 42
together a like section-modulus of 117.8 (Table IV, page 354).

uckling strength of the web, and must not be used unless the tiffened.* In such cases it will generally be advisable to select a m with a thicker web. To use these tables for other spans, or for bution of the loading, see explanation, page 566. To use these tables not secured against yielding laterally, see Lateral Deflection, nd 670.

graphs and accompanying foot-note, page 567, relating to web-buckling of

Table XII. Safe Uniform Loads in Tons of 2 000 Pounds for Be
Girder Beams

Beams secured against yielding sidewise

Span, ft	30-in G		Add for each lb increase in weight	28-in G		Add for each lb increase in weight	26-in G	
	G30 a	G30		G28 a	G28		G26 a	G26
	200 lb	180 lb		180 lb	165 lb		160 lb	150 lb
18	180.75	161.87	0.44	153.75	138.89	0.41	128.11	117.47
19	171.24	153.35	0.41	145.66	131.58	0.39	121.37	111.29
20	162.68	145.68	0.39	138.38	125.00	0.37	115.30	105.73
21	154.93	138.74	0.37	131.79	119.05	0.35	109.81	100.69
22	147.89	132.44	0.36	125.80	113.64	0.33	104.82	96.11
23	141.46	126.68	0.34	120.33	108.70	0.32	100.26	91.93
24	135.56	121.40	0.33	115.31	104.17	0.31	96.08	88.10
25	130.14	116.55	0.31	110.70	100.00	0.29	92.24	84.58
26	125.14	112.06	0.30	106.44	96.16	0.28	88.69	81.32
27	120.50	107.91	0.29	102.50	92.60	0.27	85.41	78.31
28	116.20	104.06	0.28	98.84	89.29	0.26	82.36	75.52
29	112.19	100.47	0.27	95.43	86.21	0.25	79.52	72.91
30	108.45	97.12	0.26	92.25	83.34	0.24	76.87	70.48
31	104.95	93.99	0.25	89.27	80.65	0.24	74.39	68.21
32	101.67	91.05	0.25	86.48	78.13	0.23	72.06	66.08
33	98.59	88.29	0.24	83.86	75.76	0.22	69.88	64.07
34	95.69	85.70	0.23	81.40	73.53	0.22	67.82	62.19
35	92.96	83.25	0.22	79.07	71.43	0.21	65.88	60.41
36	90.38	80.93	0.22	76.88	69.45	0.20	64.05	58.73
37	87.93	78.75	0.21	74.80	67.57	0.20	62.32	57.15
38	85.62	76.67	0.21	72.83	65.79	0.19	60.68	55.64
39	83.42	74.71	0.20	70.96	64.10	0.19	59.13	54.22
40	81.34	72.84	0.20	69.19	62.50	0.18	57.65	52.86
41	79.35	71.06	0.19	67.50	60.98	0.18	56.24	51.57
42	77.47	69.37	0.19	65.89	59.53	0.17	54.90	50.34
43	75.66	67.76	0.18	64.36	58.14	0.17	53.63	49.17
44	73.94	66.22	0.18	62.90	56.82	0.17	52.41	48.06
45	72.30	64.75	0.17	61.50	55.56	0.16	51.24	46.99
46	70.73	63.34	0.17	60.16	54.35	0.16	50.13	45.97
47	69.22	61.99	0.17	58.88	53.19	0.16	49.06	44.9
48	67.78	60.70	0.16	57.66	52.09	0.15	48.04	44.0

Safe loads given include weight of beam
Maximum fiber-stress, 16 000 lb per sq in
The section-numbers are given for convenience in identification an

ble XII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem Girder Beams

Beams secured against yielding sidewise

	24-in G		Add for each lb increase in weight	20-in G		Add for each lb increase in weight	18-in G	Add for each lb increase in weight
	G24 a	G24		G20 a	G20		G18	
	140 lb	120 lb		140 lb	112 lb		92 lb	
	155.61	133.60	0.52	130.43	104.09	0.44	78.59	0.39
	143.64	123.33	0.48	120.40	96.09	0.40	72.54	0.36
	133.38	114.52	0.45	111.80	89.23	0.37	67.36	0.34
	124.48	106.88	0.42	104.34	83.28	0.35	62.87	0.31
	116.71	100.20	0.39	97.82	78.07	0.33	58.94	0.29
	109.84	94.31	0.37	92.07	73.48	0.31	55.47	0.28
	103.74	89.07	0.35	86.95	69.40	0.29	52.39	0.26
	98.28	84.38	0.33	82.38	65.74	0.28	49.63	0.25
	93.37	80.16	0.31	78.26	62.46	0.26	47.15	0.24
	88.92	76.35	0.30	74.53	59.48	0.25	44.91	0.22
	84.88	72.88	0.29	71.14	56.78	0.24	42.87	0.21
	81.19	69.71	0.27	68.05	54.31	0.23	41.00	0.20
	77.80	66.80	0.26	65.22	52.05	0.22	39.29	0.20
	74.69	64.15	0.25	62.61	49.97	0.21	37.72	0.19
	71.82	61.66	0.24	60.20	48.04	0.20	36.27	0.18
	69.16	59.38	0.23	57.97	46.26	0.19	34.93	0.17
	66.69	57.26	0.22	55.90	44.61	0.19	33.68	0.17
	64.39	55.29	0.22	53.97	43.07	0.18	32.52	0.16
	62.24	53.44	0.21	52.17	41.64	0.17	31.43	0.16
	60.24	51.72	0.20	50.49	40.30	0.17	30.42	0.15
	58.35	50.10	0.20	48.91	39.04	0.16	29.47	0.15
	56.58	48.58	0.19	47.43	37.85	0.16	28.58	0.14
	54.92	47.15	0.18	46.04	36.74	0.15	27.74	0.14
	53.35	45.81	0.18	44.72	35.69	0.15	26.94	0.13
	51.87	44.54	0.17	43.48	34.70	0.15	26.20	0.13
	50.47	43.33	0.17	42.30	33.76	0.14	25.49	0.13
	49.14	42.19	0.17	41.19	32.87	0.14	24.82	0.12
	47.88	41.11	0.16	40.13	32.03	0.13	24.18	0.12
40	46.68	40.08	0.16	39.13	31.23	0.13	23.58	0.12

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per in

Loads given above the heavy lines are greater than safe loads for web-crippling. e paragraphs and accompanying foot-note, page 567, relating to web-buckling beams

afe loads given below the lower, broken line cause deflections exceeding 1/360 the span

he section-numbers are given for convenience in identification and ordering

Table XII (Continued). Safe Uniform Loads in Tons of 2 000 Po for Bethlehem Girder Beams

Beams secured against yielding sidewise

Span, ft	15-in G			Add for each lb increase in weight	12-in G	
	G15 b	G15 a	G15		G12 a	G12
	140 lb	104 lb	73 lb		70 lb	55 lb
10	113.26	86.76	62.83	0.39	47.89	38.40
11	102.96	78.88	57.12	0.36	43.54	34.91
12	94.38	72.30	52.36	0.33	39.91	32.00
13	87.12	66.74	48.33	0.30	36.84	29.54
14	80.90	61.97	44.88	0.28	34.21	27.43
15	75.51	57.84	41.89	0.26	31.93	25.60
16	70.79	54.23	39.27	0.25	29.93	24.00
17	66.62	51.04	36.96	0.23	28.17	22.59
18	62.92	48.20	34.91	0.22	26.61	21.33
19	59.61	45.67	33.07	0.21	25.21	20.21
20	56.63	43.38	31.42	0.20	23.95	19.20
21	53.93	41.32	29.92	0.19	22.81	18.28
22	51.48	39.44	28.56	0.18	21.77	17.45
23	49.24	37.72	27.32	0.17	20.82	16.69
24	47.19	36.15	26.18	0.16	19.95	16.00
25	45.30	34.71	25.13	0.16	19.16	15.36
26	43.56	33.37	24.17	0.15	18.42	14.77
27	41.95	32.13	23.27	0.15	17.74	14.22
28	40.45	30.99	22.44	0.14	17.10	13.71
29	39.05	29.92	21.67	0.14	16.51	13.24
30	37.75	28.92	20.94	0.13	15.96	12.80
31	36.54	27.99	20.27	0.13	15.45	12.39
32	35.39	27.11	19.63	0.12	14.97	12.00
33	34.32	26.29	19.04	0.12	14.51	11.64
34	33.31	25.52	18.48	0.12	14.09	11.29
35	32.36	24.79	17.95	0.11	13.68	10.97

Safe loads given include weight of beam. Maximum fiber-stress, 16 00 sq in

Load given above the heavy line is greater than a safe load for web-c See paragraphs and accompanying foot-note, page 567, relating to web-b of beams.

Safe loads given below the lower, broken lines cause deflections excee of the span.

The section-numbers are given for convenience in identification and o

ble XII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem Girder Beams

Beams secured against yielding sidewise

	10-in G	Add for each lb increase in weight	Span, ft	9-in G	Add for each lb increase in weight	8-in G	Add for each lb increase in weight
n.	G10			G9		G8	
	44 lb			38 lb		32.5 lb	
0	26.05	0.26	5	40.50	0.47	30.51	0.42
1	23.68	0.24	6	33.75	0.39	25.42	0.35
2	21.71	0.22	7	28.93	0.34	21.79	0.30
3	20.04	0.20	8	25.31	0.29	19.07	0.26
4	18.61	0.19	9	22.50	0.26	16.95	0.23
5	17.37	0.17	10	20.25	0.23	15.25	0.21
6	16.28	0.16	11	18.41	0.21	13.87	0.19
7	15.32	0.15	12	16.88	0.20	12.71	0.17
8	14.47	0.15	13	15.58	0.18	11.73	0.16
9	13.71	0.14	14	14.47	0.17	10.90	0.15
0	13.03	0.13	15	13.50	0.16	10.17	0.14
21	12.40	0.12	16	12.66	0.15	9.53	0.13
22	11.84	0.12	17	11.91	0.14	8.97	0.12
23	11.33	0.11	18	11.25	0.13	8.47	0.12
24	10.85	0.11	19	10.66	0.12	8.03	0.11
25	10.42	0.10	20	10.13	0.12	7.63	0.10
26	10.02	0.10	21	9.64	0.11	7.26	0.10
27	9.65	0.10	22	9.21	0.11	6.93	0.09
28	9.30	0.09	23	8.80	0.10	6.63	0.09
29	8.98	0.09	24	8.44	0.10	6.36	0.08
30	8.68	0.09	25	8.10	0.09	6.10	0.08
31	8.40	0.08	26	7.79	0.09	....	....
32	8.14	0.08	27	7.50	0.09	....	....
33	7.89	0.08	28	7.23	0.08	....	....
34	7.66	0.08	29	6.98	0.08	....	....
35	7.44	0.07	30	6.75	0.07	....	....

afe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per in

oads given above the heavy lines are greater than safe loads for web-crippling. paragraphs and accompanying foot-note, page 567, relating to web-buckling eams

afe loads given below the lower, broken lines cause deflections exceeding 1/360 he span.

he section-numbers are given for convenience in identification and ordering

Table XIII. Safe Uniform Loads in Tons of 2 000 Pounds for Bethl
I Beams

Beams secured against yielding sidewise

Span, ft	30-in I	Add for each lb increase in weight	28-in I	Add for each lb increase in weight	26-in I
	B30		B28		B26
	120 lb		105 lb		90 lb
18	103.50	0.44	84.95	0.41	67.86
19	98.05	0.41	80.48	0.39	64.29
20	93.15	0.39	76.46	0.37	61.07
21	88.71	0.37	72.82	0.35	58.16
22	84.68	0.36	69.51	0.33	55.52
23	81.00	0.34	66.49	0.32	53.11
24	77.62	0.33	63.72	0.31	50.89
25	74.52	0.31	61.17	0.29	48.86
26	71.65	0.30	58.81	0.28	46.98
27	69.00	0.29	56.64	0.27	45.24
28	66.54	0.28	54.61	0.26	43.62
29	64.24	0.27	52.73	0.25	42.12
30	62.10	0.26	50.97	0.24	40.71
31	60.10	0.25	49.33	0.24	39.40
32	58.22	0.25	47.79	0.23	38.17
33	56.45	0.24	46.34	0.22	37.01
34	54.79	0.23	44.98	0.22	35.92
35	53.23	0.22	43.69	0.21	34.90
36	51.75	0.22	42.48	0.20	33.93
37	50.35	0.21	41.33	0.20	33.01
38	49.03	0.21	40.24	0.19	32.14
39	47.77	0.20	39.21	0.19	31.32
40	46.57	0.20	38.23	0.19	30.54
41	45.44	0.19	37.30	0.18	29.79
42	44.36	0.19	36.41	0.18	29.08
43	43.33	0.18	35.56	0.17	28.41
44	42.34	0.18	34.75	0.17	27.76
45	41.40	0.17	33.98	0.16	27.14
46	40.50	0.17	33.24	0.16	26.55
47	39.64	0.17	32.54	0.16	25.99
48	38.81	0.16	31.86	0.15	25.45

Safe loads given include weight of beam. Maximum fiber-stress, 16 000
sq in

The section-numbers are given for convenience in identification and or

I (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

4-in I		Add for each lb increase in weight	20-in I					Add for each lb increase in weight
a	B24		B20 a		B20			
b	73 lb		82 lb	72 lb	69 lb	64 lb	59 lb	
2	77.45	0.52	69.33	65.18	56.40	54.32	52.10	0.44
3	71.49	0.48	63.99	60.17	52.06	50.14	48.09	0.40
2	66.38	0.45	59.42	55.87	48.34	46.56	44.65	0.37
8	61.96	0.42	55.46	52.14	45.12	43.45	41.68	0.35
6	58.08	0.39	51.99	48.88	42.30	40.74	39.07	0.33
7	54.67	0.37	48.94	46.01	39.81	38.34	36.77	0.31
1	51.63	0.35	46.22	43.45	37.60	36.21	34.73	0.29
2	48.91	0.33	43.78	41.17	35.62	34.31	32.90	0.28
3	46.47	0.31	41.60	39.11	33.84	32.59	31.26	0.26
1	44.26	0.30	39.61	37.25	32.23	31.04	29.77	0.25
2	42.24	0.29	37.81	35.55	30.76	29.63	28.42	0.24
3	40.41	0.27	36.17	34.01	29.42	28.34	27.18	0.23
1	38.72	0.26	34.66	32.59	28.20	27.16	26.05	0.22
5	37.17	0.25	33.28	31.29	27.07	26.07	25.01	0.21
2	35.74	0.24	32.00	30.08	26.03	25.07	24.04	0.20
1	34.42	0.23	30.81	28.97	25.07	24.14	23.15	0.19
1	33.19	0.22	29.71	27.93	24.17	23.28	22.33	0.19
0	32.05	0.22	28.69	26.97	23.34	22.48	21.56	0.18
9	30.98	0.21	27.73	26.07	22.56	21.73	20.84	0.17
5	29.98	0.20	26.84	25.23	21.83	21.03	20.17	0.17
8	29.04	0.20	26.00	24.44	21.15	20.37	19.54	0.16
8	28.16	0.19	25.21	23.70	20.51	19.75	18.94	0.16
4	27.33	0.19	24.47	23.00	19.90	19.17	18.39	0.15
5	26.55	0.18	23.77	22.35	19.34	18.62	17.86	0.15
1	25.82	0.17	23.11	21.73	18.80	18.11	17.37	0.15
1	25.12	0.17	22.48	21.14	18.29	17.62	16.90	0.14
6	24.46	0.17	21.89	20.58	17.81	17.15	16.45	0.14
4	23.83	0.16	21.33	20.06	17.35	16.71	16.03	0.13
7	23.23	0.16	20.80	19.55	16.92	16.30	15.63	0.13

given include weight of beam. Maximum fiber-stress, 16 000 lb per

en above the heavy lines are greater than safe loads for web-crippling.
phs and accompanying foot-note, page 567, relating to web-buckling

n-numbers are given for convenience in identification and ordering

Table XIII (Continued). Safe Uniform Loads in Tons of 2 000 for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	18-in I			Add for each lb increase in weight	15-in I			
	B18				B15 b	B15 a	B15	
	59 lb	54 lb	48.5 lb		71 lb	54 lb	46 lb	41 lb
12	43.62	41.58	39.42	0.39	47.18	36.15	28.73	27.06
13	40.26	38.38	36.39	0.36	43.55	33.37	26.52	24.98
14	37.39	35.64	33.79	0.34	40.44	30.99	24.62	23.19
15	34.90	33.26	31.54	0.31	37.75	28.92	22.98	21.65
16	32.71	31.18	29.56	0.29	35.39	27.11	21.55	20.30
17	30.79	29.35	27.83	0.28	33.30	25.52	20.28	19.10
18	29.08	27.72	26.28	0.26	31.45	24.10	19.15	18.04
19	27.55	26.26	24.90	0.25	29.80	22.83	18.14	17.09
20	26.17	24.95	23.65	0.24	28.31	21.69	17.24	16.24
21	24.93	23.76	22.53	0.22	26.96	20.66	16.42	15.46
22	23.79	22.68	21.50	0.21	25.74	19.72	15.67	14.76
23	22.76	21.70	20.57	0.21	24.62	18.86	14.99	14.12
24	21.81	20.79	19.71	0.20	23.59	18.07	14.36	13.53
25	20.94	19.96	18.92	0.19	22.65	17.35	13.79	12.99
26	20.13	19.19	18.19	0.18	21.78	16.68	13.26	12.49
27	19.39	18.48	17.52	0.17	20.97	16.07	12.77	12.03
28	18.69	17.82	16.89	0.17	20.22	15.49	12.31	11.60
29	18.05	17.21	16.31	0.16	19.52	14.96	11.89	11.20
30	17.45	16.63	15.77	0.16	18.87	14.46	11.49	10.82
31	16.88	16.10	15.26	0.15	18.26	13.99	11.12	10.47
32	16.36	15.59	14.78	0.15	17.69	13.56	10.77	10.15
33	15.86	15.12	14.33	0.14	17.16	13.15	10.45	9.84
34	15.40	14.68	13.91	0.14	16.65	12.76	10.14	9.55
35	14.96	14.26	13.52	0.13	16.18	12.39	9.85	9.28
36	14.54	13.86	13.14	0.13	15.73	12.05	9.58	9.02
37	14.15	13.49	12.78	0.13	15.30	11.72	9.32	8.78
38	13.77	13.13	12.45	0.12	14.90	11.42	9.07	8.55
39	13.42	12.79	12.13	0.12	14.52	11.12	8.84	8.33
40	13.09	12.47	11.83	0.12	14.15	10.84	8.62	8.12

Safe loads given include weight of beam. Maximum fiber-stress, sq in

Load given above the heavy line is greater than safe load for w See paragraphs and accompanying foot-note, page 567, relating to w of beams

Safe loads given below the broken lines cause deflections exceeding span

The section-numbers are given for convenience in identification an

able XIII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

an,	12-in I			Add for each lb increase in weight	10-in I		Add for each lb increase in weight
	B12 *a*	B12			B10		
	36 lb	32 lb	28.5 lb		28.5 lb	23.5 lb	
9	26.59	22.57	21.36	0.35	15.95	14.57	0.29
0	23.93	20.31	19.22	0.31	14.35	13.11	0.26
1	21.76	18.46	17.47	0.29	13.05	11.92	0.24
2	19.94	16.92	16.02	0.26	11.96	10.92	0.22
3	18.41	15.62	14.79	0.24	11.04	10.08	0.20
4	17.09	14.51	13.73	0.22	10.25	9.36	0.19
5	15.95	13.54	12.81	0.21	9.57	8.74	0.17
6	14.96	12.69	12.01	0.20	8.97	8.19	0.16
7	14.08	11.95	11.31	0.19	8.44	7.71	0.15
8	13.30	11.28	10.68	0.17	7.97	7.28	0.15
9	12.60	10.69	10.12	0.17	7.55	6.90	0.14
0	11.97	10.15	9.61	0.16	7.18	6.55	0.13
1	11.40	9.67	9.15	0.15	6.84	6.24	0.12
2	10.88	9.23	8.74	0.14	6.52	5.96	0.12
3	10.41	8.83	8.36	0.14	6.24	5.70	0.11
4	9.97	8.46	8.01	0.13	5.98	5.46	0.11
5	9.57	8.12	7.69	0.13	5.74	5.24	0.10
6	9.20	7.81	7.39	0.12	5.52	5.04	0.10
7	8.86	7.52	7.12	0.12	5.32	4.86	0.10
8	8.55	7.25	6.86	0.11	5.13	4.68	0.09
9	8.25	7.00	6.63	0.11	4.95	4.52	0.09
0	9.98	6.77	6.41	0.11	4.78	4.37	0.09
1	7.72	6.55	6.20	0.10	....	....	....
2	7.48	6.35	6.01	0.10	....	....	....
3	7.25	6.15	5.82	0.10	....	....	....
4	7.04	5.97	5.65	0.09	....	....	....
5	6.84	5.80	5.49	0.09	....	....	....

afe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per in

afe loads given below the broken lines cause deflections exceeding 1/360 of the n

he section-numbers are given for convenience in identification and ordering

Table XIII (Continued). Safe Uniform Loads in Tons of 2 000
for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	9-in I		Add for each lb increase in weight	8-in I	
	B9			B8	
	24 lb	20 lb		19.5 lb	17.5 lb
5	21.83	20.18	0.47	16.16	15.30
6	18.19	16.81	0.39	13.46	12.75
7	15.60	14.41	0.34	11.54	10.93
8	13.65	12.61	0.29	10.10	9.57
9	12.13	11.21	0.26	8.98	8.50
10	10.92	10.09	0.24	8.08	7.65
11	9.92	9.17	0.21	7.34	6.96
12	9.10	8.41	0.20	6.73	6.38
13	8.40	7.76	0.18	6.21	5.89
14	7.80	7.21	0.17	5.77	5.47
15	7.28	6.73	0.16	5.39	5.10
16	6.82	6.31	0.15	5.05	4.78
17	6.42	5.93	0.14	4.75	4.50
18	6.07	5.61	0.13	4.49	4.25
19	5.75	5.31	0.13	4.25	4.03
20	5.46	5.04	0.12	4.04	3.83
21	5.20	4.80	0.11	3.85	3.64
22	4.96	4.59	0.11	3.67	3.48
23	4.75	4.39	0.10	3.51	3.33
24	4.55	4.20	0.10	3.37	3.19
25	4.37	4.04	0.10	3.23	3.06
26	4.20	3.88	0.09	....	....
27	4.04	3.74	0.09	....	....
28	3.90	3.60	0.09	....	....
29	3.76	3.48	0.08	....	....
30	3.64	3.36	0.08	....	....

Safe loads given include weight of beam. Maximum fiber-stress, 16 sq in

Safe loads given below the broken lines cause deflections exceeding span

The section-numbers are given for convenience in identification and

s. The beam girders in Table XIV, pages 605–6, have about twice ing capacity of the single beams of which they are built.

XIV and XV give the SAFE LOADS for the SINGLE AND DOUBLE-BEAM commonly used. The values given in the tables are founded upon ents of inertia of the various sections, deductions being made for the s in both flanges. In Table XIV, taken by permission from Carnegie's ompanion, the safe loads are based upon a fiber-stress for flange-bend- 000 lb per sq in, and in Table XV, retained from the former edition of Pocket-Book, upon a fiber-stress of 13 000 lb per sq in. For other fiber- s 14 000 or 15 000 lb per sq in, the safe loads in Tables XIV or XV ecreased or increased by PROPORTION as the LOADS vary as the FIBER- †

bles of riveted plate girders, see Chapter XX.

litors decided to retain Table XV for the safe uniformly distributed loads for el-beam box girders, based upon a bending fiber-stress of 13 000 lb per sq in. s table for fiber-stresses of 14 000, 15 000 or 16 000 lb per sq in, divide the ds by 13 and multiply the quotients by 14, 15 or 16, respectively, for the safe e required fiber-stress. In regard to Table XV, Mr. Kidder said, in the editions of this pocket-book, "in order to amply compensate for the deteriora- metal around the rivet-holes from punching, and also because these girders ten used to support permanent loads, such as brick or stone walls, the maximum [for riveted double-beam girders] was limited to 13 000 lb per sq in, although ght to state that most of the latest handbooks of the steel-manufacturers give safe loads for such girders based upon a fiber-stress of 15 000 lb per sq in. r advises that for loads of masonry, which usually come very close to the loads, and which are constantly applied, the girders be not loaded beyond the en in the following tables (that is, based upon 13 000 lb per sq in), while for oor-loads, which seldom reach the estimated loads, an addition of ⅙th may be he values given in the tables."

fabricated of single steel **I** beams and plates riveted to the upper and lower shown in Table XIV, are not often used to support masonry walls, because of ively narrow flange-width and lack of lateral stiffness. In case they are used masonry walls and are not thoroughly braced laterally, it is recommended that oads be reduced as explained, from those given in Table XIV, to agree with ess of 13 000 or 14 000 lb per sq in, according to the span, bracing, character

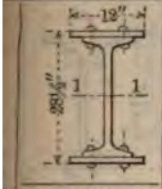
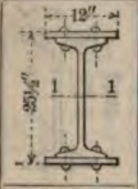
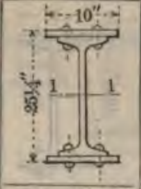
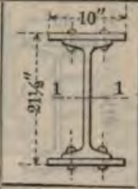
next column, the result is 21.81 tons for the safe net load, which
than required. From the following column of the table it is
creasing the thickness of the flange-plates $\frac{1}{16}$ in it is safe to add
allowable load. This will more than make up the difference.
quired DOUBLE-BEAM GIRDER will be composed of two 12-in
and two 14 by $\frac{9}{16}$-in steel flange-plates.

A SINGLE-BEAM GIRDER (according to Table XIV, page 606)
one 15-in 42-lb **I** beam and two 8 by $\frac{1}{2}$-in flange-plates will
lb per sq ft, 49 000 lb over a span of 25 ft, and as it is lighter,
69.2 lb per ft to the others 113.6 lb, it would be more
DOUBLE-BEAM GIRDER is, however, more suitable in this
13-in wall should have a wider bearing than 8 in, and, also, the
be decreased from the tabular load to correspond to a
14 000 lb per sq in because of the nature of the loading, the
or, what amounts to the same thing, the strength of the
increased to correspond to the decreased fiber-stress. (See
A 49 000-lb load at 16 000 lb per sq in fiber-stress corresponds
$\frac{16}{13}$ = 60 307-lb load at 13 000 fiber-stress, as far as selecting a
girder from table is concerned. A SINGLE-BEAM GIRDER
posed of one 15-in 60-lb **I** beam and two 9 by $\frac{5}{8}$-in
68 000 lb and weighs only 98.3 lb per ft. Therefore, as far as
cerned, to suit the conditions of loading, this would be the
GIRDER to use, and it would also be cheaper than the
determined by Table XV; but the width of bearing for the
only 9 in compared to 14 in with the DOUBLE-BEAM GIRDER.

negie, 1903, 15 000, $1\frac{3}{16}$-in rivet-holes deducted; Carnegie, 1915, no
15 000, $1\frac{3}{16}$-in rivet-holes deducted; Passaic, 15 000, $1\frac{3}{16}$ or $1\frac{5}{16}$-in
Kidder, previous and new editions, 13 000, $1\frac{5}{16}$-in rivet-holes deducted.
SINGLE-WEB, PLATE-AND-ANGLE GIRDERS (see Chapter XX): Carnegie,
$1\frac{3}{16}$-in rivet-holes deducted; Carnegie, 1915, 16 000, based upon
gross area of cross-section; Cambria, 15 000, $1\frac{3}{16}$ or $1\frac{5}{16}$-in
Passaic, 15 000, $1\frac{3}{16}$ or $1\frac{5}{16}$-in rivet-holes deducted; Kidder, previous
and 13 000 for flanges, $1\frac{3}{16}$ or $1\frac{5}{16}$-in rivet-holes deducted (also
tables). For RIVETED MULTIPLE-WEB, PLATE-AND-ANGLE GIRDERS (see

Table XIV.* Safe Uniform Loads in Units of 1 000 Pounds for Riveted Steel-Beam Girders

Maximum bending stress, 16 000† lb per sq in

								Coefficients of deflection
27-in 83-lb beam 12 by ¾-in plates		24-in 80-lb beam 12 by ¾-in plates		24-in 80-lb beam 10 by ⅝-in plates		20-in 80-lb beam 10 by ¾-in plates		
Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	
317	15.2	270	12.3	224	10.1	204	8.4	3.72
297	14.3	253	11.5	210	9.5	191	7.9	4.24
280	13.4	238	10.9	198	9.0	180	7.4	4.78
264	12.7	225	10.3	187	8.4	170	7.0	5.36
250	12.0	213	9.7	177	8.0	161	6.6	5.98
238	11.4	203	9.2	168	7.6	153	6.3	6.62
227	10.9	193	8.8	160	7.2	146	6.0	7.30
216	10.4	184	8.4	153	6.9	139	5.7	8.01
207	9.9	176	8.0	146	6.6	133	5.5	8.76
198	9.5	169	7.7	140	6.3	127	5.3	9.53
190	9.1	162	7.4	135	6.1	122	5.0	10.35
183	8.8	156	7.1	129	5.9	118	4.8	11.19
176	8.4	150	6.8	125	5.6	113	4.7	12.07
170	8.1	148	6.6	120	5.4	109	4.5	12.98
164	7.9	140	6.4	116	5.2	105	4.3	13.92
159	7.6	135	6.2	112	5.1	102	4.2	14.90
153	7.4	131	6.0	109	4.9	99	4.1	15.91
149	7.1	127	5.8	105	4.8	96	3.9	16.95
144	6.9	123	5.6	102	4.6	93	3.8	18.03
140	6.7	119	5.4	99	4.5	90	3.7	19.13
136	6.5	116	5.3	96	4.3	87	3.6	20.28
42.41 in^2 446.0 in^3 144.2 lb per ft		41.32 in^2 380.0 in^3 141.2 lb per ft		35.82 in^2 315.5 in^3 122.5 lb per ft		38.73 in^2 286.7 in^3 131.0 lb per ft		....

fe loads above the heavy, horizontal lines exceed the resistance of the web, and ers should be provided with stiffeners; for limiting conditions, see explanatory s on page 567. See Pocket Companion, 1915 for 13 and 14-ft spans

eights given for girders do not include stiffeners, rivet-heads or other details

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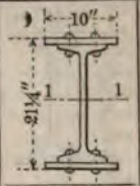
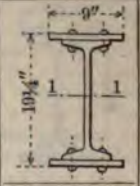
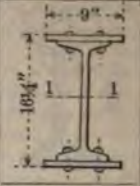
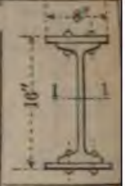
rom Pocket Companion, 1913 Edition, Carnegie Steel Company, Pittsburgh, Pa.

e paragraph on Riveted Single-Beam and Double-Beam Girders, page 603, and the te for same regarding fiber-stresses.

is the section-modulus or section-*factor of the cross-section with reference to the* 1.

Table XIV* (Continued). Safe Uniform Loads in Units of 1 000 P for Riveted Steel-Beam Girders

Maximum bending stress, 16 000† lb per sq in

Span, ft								
	20-in 65-lb beam, 10 by ⅝-in plates		18-in 55-lb beam, 9 by ⅝-in plates		15-in 60-lb beam, 9 by ⅝-in plates		15-in 42-lb bea 8 by ½-in plates	
	Safe loads	Increase in safe loads for $\frac{1}{16}$-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for $\frac{1}{16}$-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for $\frac{1}{16}$-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for $\frac{1}{16}$-in increase in thick- ness of flange-plates
10	251	12.7	196	10.3	170	8.5	123	7.6
11	228	11.6	178	9.4	155	7.7	112	6.9
12	209	10.6	164	8.6	142	7.1	102	6.4
13	193	9.8	151	7.9	131	6.5	95	5.9
14	179	9.1	140	7.4	122	6.1	88	5.5
15	167	8.5	131	6.9	113	5.7	82	5.1
16	157	8.0	123	6.5	106	5.3	77	4.8
17	148	7.5	115	6.1	100	5.0	72	4.5
18	139	7.1	109	5.7	95	4.7	68	4.2
19	132	6.7	103	5.4	90	4.5	65	4.0
20	125	6.4	98	5.2	85	4.3	61	3.8
21	119	6.1	93	4.9	81	4.0	59	3.6
22	114	5.8	89	4.7	77	3.9	56	3.5
23	109	5.5	85	4.5	74	3.7	53	3.3
24	105	5.3	82	4.3	71	3.5	51	3.2
25	100	5.1	79	4.1	68	3.4	49	3.1
26	97	4.9	76	4.0	65	3.3	47	2.9
27	93	4.7	73	3.8	63	3.1	46	2.8
28	90	4.6	70	3.7	61	3.0	44	2.7
29	87	4.4	68	3.6	59	2.9	42	2.6
30	84	4.2	65	3.4	57	2.8	41	2.5
Area	31.58 in²		27.18 in²		28.92 in²		20.48 in²	
I/c_{1-1}‡	235.2 in³		184.1 in³		159.5 in³		115.3 in³	
Wgt	107.5 lb per ft		93.3 lb per ft		98.3 lb per ft		69.2 lb per ft	

Safe loads above the heavy, horizontal lines exceed the resistance of the girders should be provided with stiffeners; for limiting conditions, see exp notes on page 567. See Pocket Companion, 1915 for 9-ft. span

Weights given for girders do not include stiffeners, rivet-heads or othe

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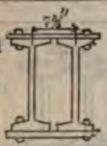
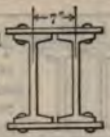
* From Pocket Companion, 1913 Edition, Carnegie Steel Company, Pittsb

† See paragraph on Riveted Single-Beam and Double-Beam Girders, page foot-note for same regarding fiber-stresses.

‡ I/c is the section-modulus or section-factor of the cross-section with refe axis 1–1.

Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

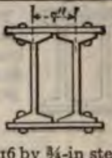
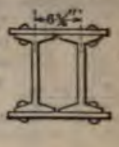
wo 20-in steel **I** beams and two 16 by ¾-in steel plates

teel es, ¾		Two 20-in beams, 80.0 lb per ft	Two steel plates, 16 by ¾ in		Two 20-in beams, 65.0 lb per ft	Increase in weight of girder for ¹⁄₁₆-in increase in thickness of flange-plates
e s. mly ib- (in- ng t of r), s of lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for ¹⁄₁₆-in increase in thickness of flange-plates	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for ¹⁄₁₆-in increase in thickness of flange-plates	
67	1.22	7.22	176.72	1.06	7.34	0.03
51	1.34	6.56	160.66	1.16	6.68	0.04
39	1.46	6.02	147.26	1.27	6.12	0.04
60	1.58	5.56	135.95	1.37	5.65	0.04
64	1.70	5.16	126.24	1.48	5.25	0.05
12	1.83	4.81	117.82	1.58	4.90	0.05
80	1.95	4.51	110.45	1.69	4.59	0.05
47	2.07	4.25	103.96	1.79	4.32	0.06
94	2.19	4.01	98.18	1.90	4.08	0.06
10	2.31	3.80	93.01	2.01	3.86	0.06
83	2.43	3.61	88.36	2.11	3.67	0.07
08	2.56	3.44	84.15	2.22	3.50	0.07
77	2.68	3.28	80.33	2.32	3.34	0.07
82	2.80	3.14	76.84	2.43	3.19	0.08
20	2.92	3.01	73.64	2.53	3.06	0.08
87	3.04	2.89	70.69	2.64	2.94	0.08
80	3.16	2.78	67.97	2.75	2.82	0.09
96	3.29	2.68	65.46	2.85	2.72	0.09
32	3.41	2.58	63.12	2.96	2.62	0.09
86	3.53	2.49	60.94	3.06	2.53	0.10
56	3.65	2.41	58.91	3.17	2.45	0.10
41	3.77	2.33	57.01	3.27	2.37	0.10
41	3.89	2.26	55.22	3.38	2.29	0.11
51	4.02	2.19	53.56	3.48	2.22	0.11
73	4.14	2.12	51.98	3.59	2.16	0.11
05	4.26	2.06	50.50	3.70	2.10	0.12
46	4.38	2.01	49.09	3.80	2.04	0.12
96	4.50	1.95	47.77	3.91	1.98	0.12
54	4.62	1.90	46.51	4.01	1.93	0.13
20	4.75	1.85	45.32	4.12	1.88	0.13

es are based on a maximum fiber-stress of 13 000 lb per sq in, rivet-holes leducted. See paragraph on Riveted Single-Beam and Double-Beam and the foot-note for same regarding *fiber-stresses.* ***Weights of girders*** gths, *center to center of bearings.*

Table XV (Continued). Safe Uniform Loads in Tons of 2 000 Po
for Riveted Steel-Beam Box Girders

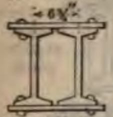
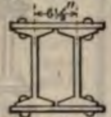
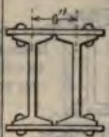
Two 18-in steel **I** beams and two 16 by ¾-in steel plates

Distance, center to center of bearings, ft	 Two 18-in beams, 70 lb per ft Two 16 by ¾-in steel plates			Two 18-in beams, 55 lb per ft  Two 16 by ¾-in steel plates			
	Safe loads in tons, including weight of girder	Weight of girder, lb	Add to safe loads for 1/16-in increase in thickness of plates	Safe loads in tons, including weight of girder	Weight of girder, lb	Add to safe loads for 5 pounds increase in weight of beam	Add t safe lo for 1/16- increas in thic ness o plates
12	132.2	2 712	5.43	123.0	2 352	2.81	5.43
13	122.0	2 933	5.01	113.5	2 548	2.61	5.01
14	113.3	3 164	4.66	105.3	2 744	2.43	4.66
15	105.7	3 390	4.35	98.3	2 940	2.27	4.35
16	99.1	3 616	4.07	92.2	3 136	2.12	4.07
17	93.3	3 842	3.83	86.8	3 332	2.00	3.83
18	88.1	4 068	3.62	82.0	3 528	1.90	3.62
19	83.5	4 294	3.43	77.6	3 724	1.80	3.43
20	79.3	4 520	3.26	73.8	3 920	1.70	3.26
21	75.5	4 746	3.10	70.2	4 116	1.62	3.10
22	72.1	4 972	2.96	67.0	4 312	1.54	2.96
23	69.0	5 198	2.83	64.1	4 508	1.47	2.83
24	66.1	5 424	2.72	61.5	4 704	1.41	2.72
25	63.5	5 650	2.61	59.0	4 900	1.36	2.61
26	61.0	5 876	2.51	56.7	5 096	1.30	2.51
27	58.8	6 102	2.41	54.6	5 292	1.26	2.41
28	56.6	6 328	2.33	52.7	5 488	1.21	2.33
29	54.7	6 554	2.25	50.9	5 684	1.17	2.25
30	52.9	6 780	2.17	49.2	5 880	1.13	2.17
31	51.8	7 006	2.10	47.6	6 076	1.10	2.10
32	49.6	7 232	2.04	46.1	6 272	1.06	2.04
33	48.1	7 458	1.98	44.7	6 468	1.03	1.98
34	46.7	7 684	1.92	43.4	6 664	1.00	1.92
35	45.3	7 910	1.86	42.1	6 860	0.97	1.86
36	44.1	8 136	1.81	41.0	7 056	0.94	1.81
37	42.9	8 362	1.76	39.9	7 252	0.92	1.76
38	41.2	8 588	1.72	38.8	7 448	0.90	1.72

The above values are based on a maximum fiber-stress of 13 000 lb per sq
in both flanges deducted. See paragraph on Riveted Single-Beam and
Girders, page 603, and the foot-note for same regarding fiber-stresses. Weig
correspond to lengths, center to center of bearings.

ble XV (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

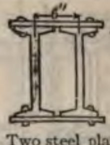
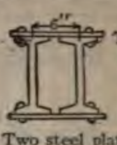
Two 15-in steel **I** beams and two 14 by ⅝-in steel plates

Two 15-in beams, 75.0 lb per ft  Steel Plates, 14 by ⅝ in		Two 15-in beams, 60.0 lb per ft  Steel Plates, 14 by ⅝ in		 Two 15-in beams, 42.0 lb per ft Steel Plates, 14 by ⅝ in		Increase in safe load for 1/16-in increase in thickness of flange-plates	Increase in weight of girder for 1/16-in increase in thickness of flange-plates
Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb		
122.33	1.06	111.01	0.91	90.29	0.72	4.63	0.03
111.21	1.17	100.92	1.00	82.08	0.79	4.21	0.03
101.95	1.27	92.51	1.09	75.24	0.86	3.86	0.03
94.10	1.38	85.40	1.18	69.45	0.93	3.57	0.04
87.38	1.48	79.30	1.27	64.50	1.00	3.31	0.04
81.56	1.59	74.01	1.36	60.19	1.08	3.09	0.04
76.46	1.70	69.38	1.45	56.43	1.15	2.90	0.05
71.96	1.80	65.30	1.54	53.11	1.22	2.72	0.05
67.96	1.91	61.67	1.63	50.16	1.29	2.57	0.05
64.39	2.01	58.43	1.72	47.52	1.36	2.43	0.05
61.17	2.12	55.50	1.81	45.14	1.44	2.32	0.06
58.25	2.22	52.86	1.90	42.99	1.51	2.21	0.06
55.60	2.33	50.46	2.00	41.04	1.58	2.11	0.06
53.19	2.43	48.27	2.09	39.25	1.65	2.02	0.07
50.97	2.54	46.25	2.18	37.62	1.72	1.93	0.07
48.93	2.65	44.40	2.27	36.12	1.79	1.85	0.07
47.05	2.76	42.70	2.36	34.72	1.87	1.78	0.08
45.31	2.86	41.12	2.45	33.44	1.94	1.71	0.08
43.69	2.96	39.65	2.54	32.25	2.01	1.66	0.08
42.18	3.07	38.28	2.63	31.13	2.08	1.60	0.08
40.78	3.17	37.00	2.72	30.09	2.15	1.54	0.09
39.46	3.28	35.81	2.81	29.12	2.23	1.49	0.09
38.23	3.38	34.69	2.80	28.21	2.30	1.45	0.09
37.07	3.46	33.64	2.99	27.36	2.37	1.41	0.10
35.98	3.60	32.65	3.08	26.55	2.44	1.37	0.10
34.95	3.70	31.72	3.17	25.80	2.51	1.33	0.10
33.98	3.81	30.84	3.27	25.08	2.58	1.29	0.10
33.06	3.91	30.00	3.36	24.40	2.66	1.25	0.11
32.20	4.02	29.21	3.45	23.76	2.73	1.22	0.11
31.37	4.13	28.47	3.54	23.15	2.80	1.19	0.11

above values are based on a maximum fiber-stress of 13 000 lb per sq in, rivet-holes
flanges deducted. See paragraph on Riveted Single-Beam and Double-Beam
, page 603, and the foot-note for same *regarding fiber-stresses.* ***Weights of girders***
ood to lengths, *center to center of bearings.*

Table XV (Continued). Safe Uniform Loads in Tons of 2 000 P
for Riveted Steel-Beam Box Girders

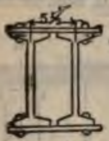
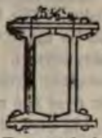
Two 12-in steel I beams and two 14 by ½-in steel plates

Distance, center to center of bearings, ft	 Two 12-in beams, 40.0 lb per ft Two steel plates, 14 by ½ in			 Two 12-in beams, 31.5 lb per ft Two steel plates, 14 by ½ in		
	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for $\frac{1}{16}$-in increase in thickness of flange-plates	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for $\frac{1}{16}$-in increase in thickness of flange-plates
10	64.94	0.65	3.75	58.08	0.57	3.81
11	59.02	0.71	3.40	52.80	0.63	3.45
12	54.12	0.78	3.12	48.40	0.68	3.17
13	49.95	0.84	2.88	44.68	0.74	2.93
14	46.39	0.91	2.68	41.48	0.80	2.72
15	43.29	0.97	2.50	38.72	0.85	2.53
16	40.59	1.04	2.34	36.30	0.91	2.38
17	38.20	1.10	2.21	34.16	0.97	2.24
18	36.08	1.17	2.08	32.27	1.03	2.11
19	34.18	1.23	1.97	30.57	1.08	2.00
20	32.47	1.30	1.87	29.04	1.14	1.90
21	30.93	1.36	1.78	27.66	1.20	1.81
22	29.52	1.43	1.70	26.40	1.25	1.73
23	28.23	1.49	1.63	25.25	1.31	1.65
24	27.06	1.56	1.56	24.20	1.37	1.58
25	25.98	1.62	1.50	23.23	1.42	1.52
26	24.98	1.69	1.44	22.34	1.48	1.46
27	24.05	1.75	1.38	21.51	1.54	1.41
28	23.19	1.82	1.34	20.74	1.60	1.36
29	22.39	1.88	1.29	20.03	1.65	1.31
30	21.65	1.95	1.25	19.36	1.71	1.27
31	20.95	2.01	1.21	18.73	1.77	1.23
32	20.29	2.08	1.17	18.15	1.82	1.19
33	19.68	2.14	1.14	17.60	1.88	1.15
34	19.10	2.21	1.10	17.08	1.94	1.12
35	18.55	2.27	1.07	16.59	1.99	1.09
36	18.04	2.34	1.04	16.13	2.05	1.06
37	17.55	2.40	1.01	15.70	2.11	1.03
38	17.09	2.47	0.99	15.28	2.17	1.00
39	16.65	2.53	0.96	14.89	2.22	0.98

The above values are based on a maximum fiber-stress of 13 000 lb per sq in, ri in both flanges deducted. See paragraph on Riveted Single-Beam and Doub Girders, page 603, and the foot-note for same regarding fiber-stresses. Weights o correspond to lengths, center to center of bearings.

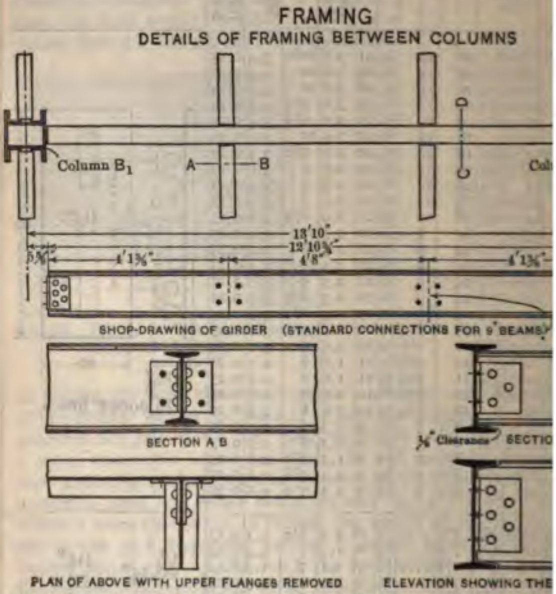
able XV (Continued). **Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders**

Two 10-in steel **I** Beams and two 12 by ½-in steel plates

is- ce, er to der ear- s, ft	 Two 10-in beams, 35.0 lb per ft Two steel plates, 12 by ½ in			 Two 10-in beams, 25.0 lb per ft Two steel plates, 12 by ½ in			Increase in weight of girder for 1/16-in increase in thickness of flange-plates
	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	
0	44.35	0.55	2.59	39.23	0.47	2.64	0.02
1	40.32	0.60	2.36	35.66	0.52	2.40	0.03
2	36.96	0.65	2.16	32.69	0.56	2.20	0.03
3	34.12	0.71	1.99	30.18	0.61	2.03	0.03
4	31.68	0.76	1.85	28.02	0.66	1.89	0.03
5	29.57	0.82	1.73	26.15	0.71	1.76	0.04
6	27.72	0.87	1.62	24.52	0.75	1.65	0.04
7	26.09	0.93	1.52	23.08	0.80	1.55	0.04
8	24.64	0.98	1.44	21.79	0.85	1.47	0.04
9	23.34	1.04	1.36	20.65	0.89	1.39	0.05
0	22.18	1.09	1.30	19.62	0.94	1.32	0.05
1	21.12	1.15	1.23	18.68	0.99	1.26	0.05
2	20.16	1.20	1.18	17.83	1.04	1.20	0.05
3	19.28	1.26	1.13	17.06	1.08	1.15	0.06
4	18.48	1.31	1.08	16.35	1.13	1.10	0.06
5	17.74	1.36	1.04	15.69	1.18	1.06	0.06
6	17.06	1.42	1.00	15.09	1.22	1.02	0.06
7	16.43	1.47	0.96	14.53	1.27	0.98	0.07
8	15.84	1.53	0.93	14.01	1.32	0.94	0.07
9	15.29	1.58	0.89	13.53	1.37	0.91	0.07
0	14.78	1.64	0.86	13.08	1.41	0.88	0.07
1	14.31	1.69	0.84	12.65	1.46	0.85	0.08
2	13.86	1.75	0.81	12.26	1.51	0.82	0.08
3	13.44	1.80	0.78	11.89	1.55	0.80	0.08
4	13.04	1.86	0.76	11.54	1.60	0.78	0.08
5	12.67	1.91	0.74	11.21	1.65	0.75	0.09
6	12.32	1.96	0.72	10.90	1.70	0.73	0.09
7	11.99	2.02	0.70	10.60	1.74	0.71	0.09
8	11.67	2.07	0.68	10.32	1.79	0.69	0.09
9	11.37	2.13	0.66	10.06	1.84	0.67	0.10

e above values based on a maximum fiber-stress of 13 000 lb per sq in, rivet-holes th flanges deducted. See paragraph on Riveted Single-*Beam and Double-Beam* ers, page 603, and the foot-*note for same regarding fiber-stresses. Weights of girders* spond *to lengths, center to center of bearings.*

Gas-Pipe Separators. Separators formed of pieces of GAS-PIP
desired lengths and slipped over the bolts are often used by contr
bottom of Table XVI.) Such separators permit the beams to act i
of each other, and should not be used in any place where one bea
receive a greater load than the other; and as this condition ex
every case where two or more beams are used together, it follow
iron separators, made to fit the space between the beams," shoul
in almost every instance. As noted in Table XVI, gas-pipe may
used for 5, 4 and 3-in beams. Separators with two bolts shoul
beams 12 in or more in depth. For 12-in beams one bolt is someti
the load is light; for beams under 12 in in depth one bolt is suffic

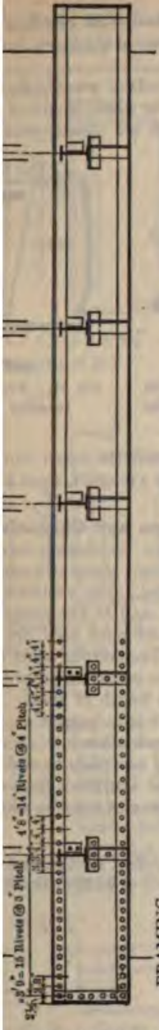

FRAMING
DETAILS OF FRAMING BETWEEN COLUMNS
Column B₁
A
B
Col
13'10"
12'10⅜"
5⅜"
4'1⅜"
4'8"
4'1⅜"
SHOP-DRAWING OF GIRDER (STANDARD CONNECTIONS FOR 9" BEAMS)
SECTION A B
⅜" Clearance
SECTIO
PLAN OF ABOVE WITH UPPER FLANGES REMOVED
ELEVATION SHOWING THE

CONNECTIONS FOR BEAMS AND GIRDERS

"Connection-angles shall in no case be less in thickness than the w
or girder to which they are fastened, nor shall the width be less than
the beam, except that no angle-knee shall be less than 2½" wide n
more than 6" wide. Web-angles, the full depth of the web, must be u
connections."

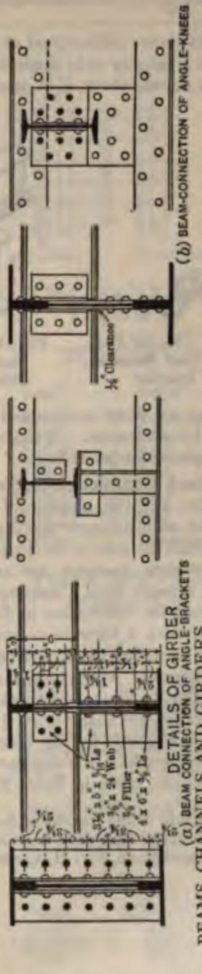
Fig. 12. Framing of Steel I Beams and Girders

Beam-Connections. Steel beams and channels are FRAMI
means of short pieces of angles, which are usually riveted to th
tail-beam and bolted to the girder. The angles are always used
each side of the beam. If the floor-beam is framed flush, either
bottom of the girder, or if two beams of the same height are fram
end of the beam supported should be COPED, or cut to fit the sha
or supporting beam. The maximum clearance-space allowed b



FRAMING.

"Beams are to be set with their lower flange flush with that of the beam girder and coped to fit it, except where the girder is on a line with a permanent partition, when it may be set 1½ in below the beams so the latter may be squared on the ends. Smaller beams are to be framed level on the bottom with these and coped. All openings in the floors, except those extending across the full width of the floor-arch, shall be framed with beams on all sides. Built-up girders are to be set with their upper flange not more than 3 in above the upper flange of the beams. Wherever necessary and required 3½ by 3½ by ⅜-in continuous L's shall be riveted to the beams, channels and girders to support the floor, roof and vault-arching."



DETAILS OF GIRDER

(*a*) BEAM CONNECTION OF ANGLE-BRACKETS

(*b*) BEAM-CONNECTION OF ANGLE-KNEES

BEAMS, CHANNELS AND GIRDERS.

. . . "Not more than ⅛ in clearance will be allowed at each end of beams and channels between girders and not more than ¼ at each end of beams, channels and girders between columns. All open holes must be true to the drawings and an error of more than 1⁄16 in in the distance from end to end between the open holes in the flanges or, a variation of more than ½ in in the length of beams supported by connection-angles, measured from back to back of same, will be sufficient cause for rejection."

Fig. 13. Framing to Riveted Plate Girder

d the web varies from 1⁄16 in in the smaller beams to ⅛ in in the igs. 12 and 13 show various details of beams framed together lers. When a floor-beam rests on top *of another beam or girder,* he beam *should be secured by means of a pair of wrought-iron*

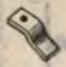


Fig. 14. Clip for Fastening Steel Beam on Top of Another

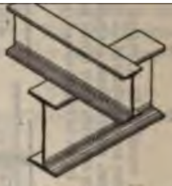


Fig. 15. Steel Beams Fastened One on the Other by Clips

Fig. 16. Frami
Joists to Ste

CLAMPS OR ANCHORS, and framed about 1 in above the upper flange
to allow for settlement. If these joists are over 3 ft apart, short leng
may be placed under each one.*

Standard Connection-Angles for I Beams and Channels.
the angles and the number of rivets used for connecting steel
somewhat with different shops and with different structural engine
there cannot be said to be a universal standard. The variations i
ent STANDARDS, however, are not very great, and as the connecti
by the Carnegie Steel Company are perhaps the most used, the a
lected them for illustration in Table XVII. The CONNECTIONS ha
portioned with a view to covering most cases occurring in ordina
with the usual relations of depth of beam to length of span. In
stances, however, where beams of short relative span-lengths are lo
full capacity, or when beams frame opposite each other into anothe
web-thickness less than 5/16 in, it may be found necessary to make
additional strength in the connections. The LIMITING SPAN-LE
at and above which the standard connection-angles may be used
safety, are also given in Table XVIII.

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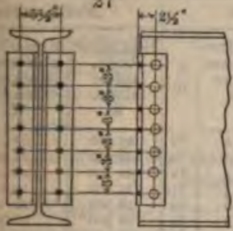
* For details of the framing of floor-beams and girders, see Chapters X
and also Professor Nolan's revised Chapters II and VII of Kidder's Building
and Superintendence, Part II, Carpenters' Work.

Table XVII.* Connections for Steel Beams

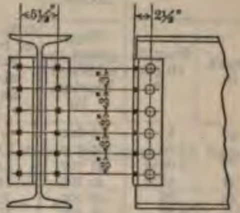
AMERICAN BRIDGE COMPANY STANDARD

27"



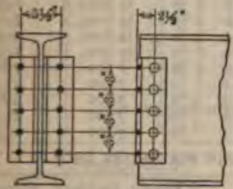
2 Ls 4"x 4"x ½"x 1' 8½"
Weight 46 lb

24"



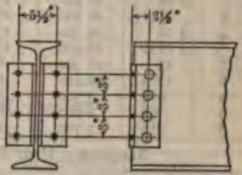
2 Ls 4"x 4"x ½"x 1' 5½"
Weight 39 lb

21"



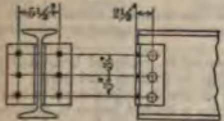
2 Ls 4"x 4"x ½"x 1' 2½"
Weight 33 lb

20", 18", 15"



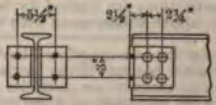
2 Ls 4"x 4"x 5/16"x 0' 11½"
Weight 23 lb

12"



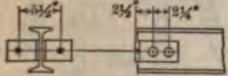
2 Ls 4"x 4"x 5/16"x 0' 8½"
Weight 17 lb

10", 9", 8"



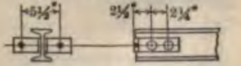
2 Ls 6"x 4"x ⅜"x 0' 5½"
Weight 13 lb

7", 6", 5"



2 Ls 6"x 4"x ⅜"x 0' 3"
Weight 7 lb

4", 3"



2 Ls 6"x 4"x ⅜"x 0' 2"
Weight 5 lb

Rivets and bolts ¾" diameter
Weights given are for ¾" shop rivets and angle-connections; about 20 per cent should be added for field-rivets or bolts

* From *Pocket Companion, 1915 Edition, Carnegie Steel Company,* Pittsburgh, Pa.

Table XVIII.* Limiting Values of Connections for Steel Bea

I beams		Value of web-connection	Values of outstanding legs of connectio				
			Field-rivets			Field-bo	
Depth, in	Weight, lb per ft	Shop-rivets in enclosed bearing, lb	¾-in rivets or turned bolts, single shear, lb	Min. allowable span, uniform load, ft	*t*, in	¾-in rough bolts, single shear, lb	Min abl un
27	83	66 800	61 900	18.4	⅝	49 500	
24	80	67 500	53 000	17.5	⅝	42 400	
	69½	52 700	53 000	16.3	⅝	42 400	
21	57½	40 200	44 200	15.5	9/16	35 300	
20	65	45 000	35 300	17.6	⅝	28 300	
18	55	41 400	35 300	13.3	⅝	28 300	
	46	29 000	35 300	15.0	½	28 300	
15	42	36 900	35 300	8.9	⅝	28 300	
	36	26 000	35 300	11.1	7/16	28 300	
12	31½	23 600	26 500	8.1	9/16	21 200	
	27½	17 200	26 500	10.3	7/16	21 200	
10	25	27 900	17 700	7.4	⅝	14 100	
	22	20 900	17 700	6.9	⅝	14 100	
9	21	26 100	17 700	5.7	⅝	14 100	
8	18	24 300	17 700	4.3	⅝	14 100	
	17½	18 900	17 700	4.4	⅝	14 100	
7	15	11 300	8 800	6.2	⅝	7 100	
6	12¼	10 400	8 800	4.4	⅝	7 100	
5	9¾	9 500	8 800	2.9	⅝	7 100	
4	7½	8 600	8 800	2.2	9/16	7 100	
3	5½	7 700	8 800	1.3	½	7 100	

ALLOWABLE UNIT STRESS IN POUNDS PER SQUARE INCH

Single shear	Rivets............shop 12 000 Rivets and turned bolts...........field 10 000 Rough bolts......field 8 000	Bearing	Rivets, enclosed.. Rivets, one side.. Rivets and turned bolts.......... Rough bolts......

t = Web-thickness, in bearing, to develop maximum allowable react beams frame opposite

Connections are figured for bearing and shear (no moment conside

The above values agree with tests made on beams under ordinary co use

Where the web is enclosed between connection-angles (enclosed beari are greater because of the increased efficiency due to friction and grip

Special connections must be used when any of the limiting condit above are exceeded, as when an end-reaction from a loaded beam is g the value of the connection of the shorter span with the beam fully load thickness of web when maximum allowable reactions are used

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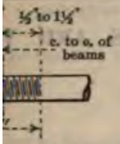
* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pit

XIX.* Lengths and Weights of Tie-Rods and Anchors for Steel Beams

AMERICAN BRIDGE COMPANY STANDARD



¾-INCH TIE-RODS

AND WEIGHTS FOR VARIOUS DISTANCES CENTER TO CENTER OF BEAMS

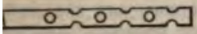
Weights include two nuts

h	Wgt	C to C	L'th	Wgt	C to C	L'th	Wgt	C to C	L'th	Wgt
in	lb	ft in	ft in	lb	ft in	ft in	lb	ft in	ft in	lb
3	2.30	1 3	1 6	2.67	1 6	1 9	3.05	1 9	2 0	3.42
3	3.80	2 3	2 6	4.17	2 6	2 9	4.55	2 9	3 0	4.92
3	5.30	3 3	3 6	5.67	3 6	3 9	6.05	3 9	4 0	6.42
3	6.80	4 3	4 6	7.17	4 6	4 9	7.55	4 9	5 0	7.92
3	8.30	5 3	5 6	8.67	5 6	5 9	9.05	5 9	6 0	9.42
3	9.80	6 3	6 6	10.17	6 6	6 9	10.55	6 9	7 0	10.92
3	11.30	7 3	7 6	11.67	7 6	7 9	12.05	7 9	8 0	12.42
3	12.80	8 3	8 6	13.17	8 6	8 9	13.55	8 9	9 0	13.92

For strength of rods, see Table II, page 388.

Anchors*

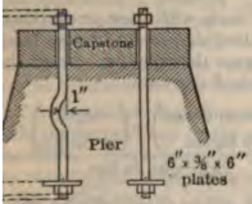
SWEDGE-BOLT



Weight includes nut

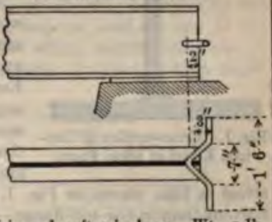
BUILT-IN ANCHOR-BOLTS

r	Length	Weight
	ft in	lb
	0 9	1.3
	1 0	2.3
	1 0	3.1
	1 3	6.1



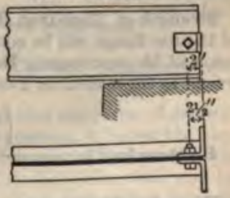
er to center of anchors is less than washer, use washer with two holes

GOVERNMENT ANCHOR



¾-in rod, 1 ft 9 in long. Wt., 3 lb

ANGLE-ANCHOR



Two angles, 6 by 4 by 7/16 by 2½ in
Weight with ¾-in bolts, 7 lb

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ring-plates, bases, etc., see Chapter XIII.

'ocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

CHAPTER XVI

STRENGTH OF CAST-IRON LINTELS AND W BEAMS

By
F. H. KINDL
CORRESPONDING MEMBER AMERICAN INSTITUTE OF ARCHITEC

1. Cast-Iron Lintels

Form of Cross-Section. Owing to the fact that the resistance o to tension is only about one-fifth of its resistance to compression, th beams most economical for wrought iron or steel would be wastef iron. The extreme brittleness of cast iron, and the danger of flaws i render it an undesirable material for resisting transverse stress. only form in which cast-iron beams are now used in building-const this country is in the shape of LINTELS for supporting brick or ston places where a flat soffit is desired, and th not to be plastered. CAST-IRON LINTEL occasionally used over store-fronts, the f lintel being paneled and molded for ar effect.

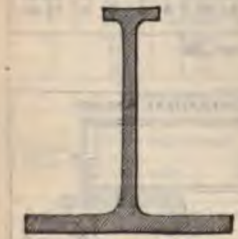


Fig. 1. Cross-section of Cast-iron Lintel of Ideal Form

Experiments on Cast-Iron Beam wrought-iron I beams were manufactu IRON BEAMS were frequently used as available ones, other than those of wood Early in the nineteenth century Eaton son, an English engineer, made a series ments with cast-iron beams, from which that the form of cross-section of a bea material which will resist the greatest stress is that shown in Fig. 1, in which there is six times more m bottom than in the top flange. The relative thicknesses of the three web, the top flange and the bottom flange, may be, with advantage, 8, respectively.

Strength of Cast-Iron Beams. If made with these proportions, of the top flange will be equal to one-third that of the bottom flang result of his experiments, Hodgkinson gave the following rule for the weight at the middle for a cast-iron beam of this form:

$$\text{Breaking-load in tons} = \frac{\begin{pmatrix}\text{area of bottom flange}\\ \text{in square inches}\end{pmatrix} \times \begin{pmatrix}\text{depth}\\ \text{in inches}\end{pmatrix} \times}{\text{clear span in feet}}$$

This rule, although largely empirical, agreed very well with the ments that were made. Structural engineers, however, use the gen las for the strength of beams, as given in Chapter XV, except that th MODULUS is found by dividing the MOMENT OF INERTIA by the dist neutral axis from the bottom of the beam, and the SAFE TENSILE S

LEXURE-FORMULA. Thus the general formula for a beam supported and with the load uniformly distributed, as given in Chapter XV,

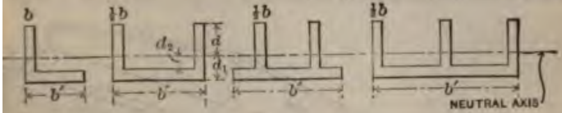
in pounds $= \frac{2}{3}\frac{I/c}{l} \times S_t$. As S_t, the safe tensile strength for cast

pe taken at 3 000 lb, this formula becomes

$$\text{Safe load in pounds} = \frac{2\,000\, I/c}{l} \tag{2}$$

er section given below,

$$I/c = \frac{\text{Moment of inertia}}{d_1}$$



NT OF INERTIA is computed by the formula (see page 335)

$$I = \frac{bd^3 + b'd_1^3 - (b' - b)d_2^3}{3} \tag{3}$$

lenotes the combined thickness of the webs, and the distances d, e measured from the NEUTRAL AXIS, which must pass through the RAVITY of the section. The center of gravity may be found by the lained in Chapter VI. This formula may be used for any of the ns when the depth does not exceed the width, and the thickness of at least equal to the thickness of the flange. In lintels with a single ell to make the thickness ¼ or ⅛ in greater than ss of the flange. For a a cross-section like that ig. 1, Formula (2) agrees with Formula (1), when safety of six is used.

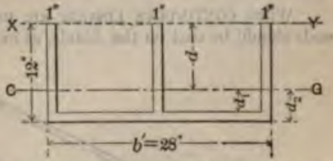


Fig. 2. Cross-section of Cast-iron Lintel with Three Webs

The following example he application of Formula quired to compute the safe ast-iron lintel having the vn in Fig. 2 and a clear span of 10 ft. The load is uniformly dis- d the thickness of the metal 1 in.

The first step is the finding of distance d, that the center of gravity, ch the neutral axis of the cross-section passes, is below the top e beam. This is found by taking the moments of the areas of the s of webs and flange about the line XY, and dividing their sum by the entire section. (See page 294.) Each web-section is 11 in a thick; hence the area of each is 11 sq in. The MOMENTS OF THE about XY will then be $3 \times 11 \times 5\tfrac{1}{2} = 181.5$

T OF THE FLANGE about $XY = 28 \times 11\tfrac{1}{2} = \underline{322}$

503.5

The area of the entire cross-section = 61 sq in

$503.5 \div 61 = 8.25 = d$ in

Then $d = 8.25$ in $d^3 = 561.5$ $b = 3$ in

$d_1 = 3.75$ in $d_1^3 = 52.7$ $b' = 28$ in

$d_2 = 2.75$ in $d_2^3 = 20.8$

The MOMENT OF INERTIA is next found by Formula (3):

$$I = \frac{3 \times 561.5 + 28 \times 52.7 - 25 \times 20.8}{3} = 880$$

$I/c = 880/3.75$. From Formula (2) the safe load = $(2\,000 \times 234.6)/12 =$
lb, or 23.4 tons.

Ends and Brackets of Cast-Iron Lintels. When a lintel, the
tion of which has the shape of an inverted T (⊥), is used over a single

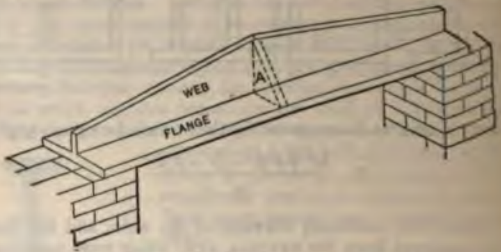


Fig. 3. Cast-iron Lintel with Tapering Web

web may be tapered towards the ends, as in Fig. 3, without affecting the
If the flange is more than 8 in wide, brackets should be cast in the middle,
A, Fig. 3.

When CONTINUOUS LINTELS are used over store-fronts or similar
ends should be cast on the lintels, as in Fig. 4, and the ends of abutting

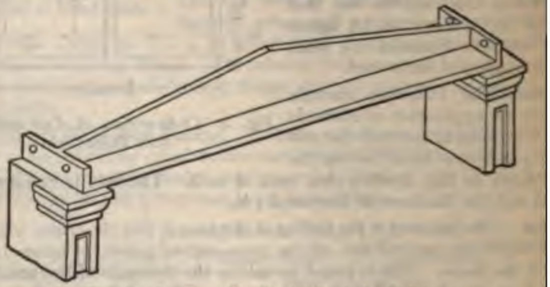


Fig. 4. Cast-iron Lintel with Ends for Bolting

bolted together. All lintels with two or three webs should have solid
necting the webs.

Tables of Strength of Cast-Iron Lintels. The tables on the
pages have been computed in accordance with Formula (2). The weight

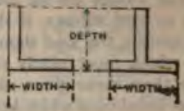
it is required to support a 12-in brick wall, 10 ft high, over an in wide, with a cast-iron lintel. At a distance of 22 in from one der, which may bring a load of 9 600 lb on the lintel, enters the hould be the dimensions of the lintel?

At 110 lb per cu ft, the wall above the lintel weighs 10 × 5½ × 110 As 22 in is one-third of the span, the concentrated load is multi- (page 632), making the load 17 088 lb. The total equivalent dis- is then 23 138 lb. Multiplying this by the span there results 3.6 tons, as the least value for the coefficient of strength *C*. From found that a 12 by 10-in lintel, 1 in thick, with one web, has a trength of 72.2; and that a 12 by 8 by 1¼-in lintel with two webs, t of strength of 69.9. A lintel with two webs is best for a 12-in polating between the values of *C* for the 1-in and 2-in thicknesses in lintel, 65.4 is found to be the value of *C* for a thickness of 1⅛ eds the required value by enough to more than compensate for the lintel itself; hence a 12 by 8 by 1⅛-in lintel with two webs

stings. Owing to the liability of flaws in the castings, cast-iron lways be carefully inspected before being accepted.

Table I. Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF



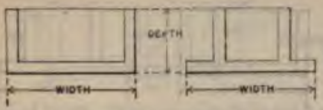
SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per
remarks, pages 622 and 623.

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet					
				5	6	7	8	9	10
6× 6	¾	26.3	15.9	3.18	2.65	2.27	1.98	1.76	1.59
	1	34.4	19.0	3.80	3.16	2.71	2.37	2.11	1.90
	1¼	42.0	21.5	4.30	3.58	3.07	2.68	2.39	2.15
7× 6	¾	28.6	17.8	3.56	2.96	2.54	2.22	1.98	1.78
	1	37.5	21.3	4.26	3.55	3.04	2.66	2.36	2.13
	1¼	45.9	24.0	4.80	4.00	3.43	3.00	2.66	2.40
7× 7	¾	31.0	22.6	4.52	3.76	3.23	2.82	2.51	2.26
	1	40.6	27.5	5.50	4.58	3.93	3.43	3.05	2.75
	1¼	49.8	31.4	6.28	5.23	4.49	3.92	3.49	3.14
8× 6	¾	31.0	19.6	3.92	3.26	2.80	2.45	2.18	1.96
	1	40.6	23.4	4.68	3.90	3.34	2.92	2.60	2.34
	1¼	49.8	26.4	5.28	4.40	3.77	3.30	2.93	2.64
8× 7	¾	33.3	25.0	5.00	4.16	3.57	3.12	2.77	2.50
	1	43.7	30.3	6.06	5.05	4.33	3.79	3.36	3.03
	1¼	53.7	34.8	6.96	5.80	4.97	4.35	3.86	3.48
8× 8	¾	35.6	30.6	6.12	5.10	4.37	3.82	3.40	3.06
	1	46.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76
	1¼	57.6	43.4	8.68	7.23	6.20	5.42	4.82	4.34
8× 9	¾	38.0	36.5	7.30	6.08	5.21	4.56	4.05	3.65
	1	50.0	45.2	9.04	7.53	6.45	5.65	5.02	4.52
	1¼	61.5	52.6	10.52	8.76	7.51	6.57	5.84	5.26
12× 6	¾	40.4	26.5	5.30	4.41	3.78	3.31	2.94	2.65
	1	53.1	31.6	6.32	5.26	4.51	3.95	3.51	3.16
	1¼	65.4	34.8	6.96	5.80	4.97	4.35	3.86	3.48
12× 8	¾	45.0	41.7	8.34	6.95	5.95	5.21	4.63	4.17
	1	59.4	51.2	10.24	8.53	7.31	6.40	5.69	5.12
	1¼	73.2	58.5	11.70	9.75	8.35	7.31	6.50	5.85
12×10	¾	49.8	58.0	11.60	9.66	8.28	7.25	6.44	5.80
	1	65.6	72.2	14.44	12.03	10.31	9.02	8.02	7.22
	1¼	81.0	83.8	16.76	13.96	11.97	10.47	9.31	8.38
12×12	¾	54.4	75.2	15.04	12.53	10.74	9.40	8.35	7.52
	1	71.9	94.8	18.96	15.80	13.54	11.85	10.53	9.48
	1¼	88.9	111.5	22.30	18.58	15.92	13.93	12.39	11.15

ble 1 (Continued). Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF



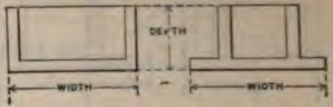
SHAPES

ads include weights of lintels. Maximum tensile stress 3 000 lb per sq in. See arks, pages 622 and 623.

ze. dth y th. n	Thickness of metal, in	Weight per foot, lb	*C*, tons	Span in feet							
				5	6	7	8	9	10	11	12
	¾	52.7	31.7	6.34	5.28	4.53	3.96	3.52	3.17	2.88	2.64
6	1	68.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76	3.42	3.13
	1¼	84.0	43.0	8.60	7.16	6.14	5.37	4.77	4.30	3.91	3.58
	¾	62.1	49.5	9.90	8.25	7.07	6.19	5.50	4.95	4.50	4.12
8	1	81.3	60.9	12.18	10.15	8.70	7.61	6.76	6.09	5.53	5.07
	1¼	99.6	69.9	13.98	11.65	9.98	8.73	7.76	6.99	6.35	5.82
	¾	57.4	35.5	7.10	5.91	5.07	4.43	3.94	3.55	3.22	2.96
6	1	75.0	42.0	8.40	7.00	6.00	5.25	4.66	4.20	3.82	3.50
	1¼	91.8	48.0	9.60	8.00	6.85	6.00	5.33	4.80	4.36	4.00
	¾	66.8	55.4	11.08	9.23	7.91	6.92	6.15	5.54	5.03	4.61
8	1	87.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81	6.19	5.67
	1¼	107.4	78.8	15.76	13.13	11.25	9.85	8.75	7.88	7.16	6.56
	¾	62.1	39.1	7.82	6.51	5.58	4.88	4.34	3.91	3.55	3.25
× 6	1	81.3	46.8	9.36	7.80	6.68	5.85	5.20	4.68	4.25	3.90
	1¼	99.6	52.9	10.58	8.81	7.55	6.61	5.88	5.29	4.81	4.40
	¾	71.5	61.4	12.28	10.23	8.77	7.67	6.82	6.14	5.58	5.11
× 8	1	93.8	74.6	14.92	12.43	10.65	9.32	8.29	7.46	6.78	6.21
	1¼	115.2	86.8	17.36	14.46	12.40	10.85	9.64	8.68	7.89	7.23
	¾	71.5	47.2	9.44	7.86	6.74	5.90	5.24	4.72	4.29	3.93
× 6	1	93.8	55.1	11.02	9.18	7.87	6.88	6.12	5.51	5.01	4.59
	1¼	115.2	62.0	12.40	10.33	8.85	7.75	6.88	6.20	5.63	5.16
	¾	80.8	72.6	14.52	12.10	10.37	9.07	8.06	7.26	6.60	6.05
× 8	1	106.2	89.5	17.90	14.91	12.78	11.18	9.94	8.95	8.13	7.45
	1¼	130.8	102.5	20.50	17.08	14.64	12.81	11.39	10.25	9.31	8.54
	¾	90.2	100.5	20.10	16.75	14.35	12.56	11.16	10.05	9.13	8.37
20×10	1	118.8	125.4	25.08	20.90	17.91	15.67	13.93	12.54	11.40	10.45
	1¼	146.5	146.8	29.36	24.46	20.97	18.35	16.31	14.68	13.34	12.23
	¾	99.6	122.6	24.52	20.43	17.51	15.32	13.62	12.26	11.14	10.21
×12	1	131.3	158.0	31.60	26.33	22.57	19.75	17.55	15.80	14.36	13.16
	1¼	162.1	189.5	37.90	31.58	27.07	23.68	21.05	18.95	17.22	15.79
	¾	90.2	83.4	16.68	13.90	11.91	10.42	9.26	8.34	7.58	6.95
24× 8	1	118.8	102.4	20.48	17.06	14.63	12.80	11.37	10.24	9.31	8.53
	1¼	146.5	117.0	*23.40*	*19.50*	*16.71*	*14.62*	*13.00*	*11.70*	10.63	9.75

Table I (Continued). Safe Distributed Loads in Tons for Cast-Iron

LINTELS OF

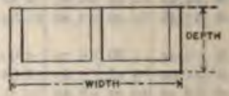


SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per
remarks, pages 622 and 623.

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet					
				5	6	7	8	9	10
24×10	¾	99.6	116.0	23.20	19.33	16.57	14.50	12.88	11.60
	1	131.3	144.4	28.88	24.06	20.63	18.05	16.04	14.44
	1¼	162.1	167.6	33.52	27.93	23.94	20.95	18.62	16.76
24×12	¾	109.0	150.4	30.08	25.06	21.48	18.80	16.71	15.04
	1	143.8	189.6	37.92	31.60	27.08	23.70	21.06	18.96
	1¼	177.7	223.0	44.60	37.16	31.85	27.87	24.77	22.30
28× 8	¾	99.6	95.5	19.10	15.91	13.64	11.93	10.61	9.55
	1	131.3	115.0	23.00	19.16	16.43	14.37	12.77	11.50
	1¼	162.1	130.5	26.10	21.75	18.64	16.31	14.50	13.05
28×10	¾	109.0	130.0	26.00	21.67	18.57	16.25	14.44	13.00
	1	143.8	164.8	32.96	27.46	23.54	20.60	18.31	16.48
	1¼	177.7	188.5	37.70	31.41	26.93	23.56	20.94	18.85
28×12	¾	118.3	162.5	32.50	27.08	23.21	20.31	18.06	16.25
	1	156.3	211.8	42.36	35.30	30.26	26.48	23.53	21.18
	1¼	193.3	252.0	50.40	42.00	36.00	31.50	28.00	25.20

LINTELS OF

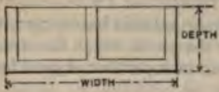


SHAPES

16× 6	¾	74.4	43.3	8.66	7.21	6.18	5.41	4.81	4.33
	1	96.9	52.4	10.48	8.73	7.48	6.55	5.82	5.24
	1¼	118.1	59.3	11.86	9.88	8.47	7.41	6.59	5.93
16× 8	¾	88.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81
	1	115.6	83.9	16.75	13.98	11.98	10.48	9.32	8.39
	1¼	141.6	97.0	19.40	16.16	13.85	12.12	10.77	9.70
20× 8	¾	97.8	80.2	16.04	13.36	11.45	10.02	8.91	8.02
	1	128.1	98.7	19.74	16.45	14.10	12.33	10.96	9.87
	1¼	157.2	113.9	22.78	18.98	16.27	14.23	12.65	11.39

(Continued). Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF



SHAPES

nclude weights of lintels. Maximum tensile stress 3 000 lb per sq in. See pages 622 and 623.

hick-ess of netal, in	Weight per foot, lb	C, tons	Span in feet							
			5	6	7	8	9	10	11	12
¾	111.9	112.0	22.40	18.66	16.00	14.00	12.44	11.20	10.18	9.33
1	146.9	139.7	27.94	23.28	19.95	17.46	15.52	13.97	12.70	11.64
1¼	180.7	163.5	32.70	27.25	23.35	20.43	18.16	16.35	14.86	13.62
¾	126.0	146.7	29.34	24.45	20.95	18.33	16.30	14.67	13.33	12.22
1	165.6	184.8	36.96	30.80	26.40	23.10	20.53	18.48	16.80	15.40
1¼	204.1	218.8	43.76	36.46	31.25	27.35	24.31	21.88	19.89	18.24
¾	107.2	91.9	18.38	15.31	13.12	11.49	10.21	9.19	8.35	7.66
1	140.6	112.8	22.56	18.80	16.11	14.10	12.53	11.28	10.25	9.40
1¼	172.6	130.2	26.04	21.70	18.57	16.27	14.47	13.02	11.83	10.85
¾	121.3	127.8	25.56	21.30	18.25	15.97	14.20	12.78	11.61	10.65
1	159.4	159.5	31.90	26.58	22.78	19.94	17.72	15.95	14.50	13.29
1¼	196.3	183.6	36.72	30.60	26.23	22.95	20.40	18.36	16.69	15.30
¾	135.3	166.6	33.32	27.76	23.80	20.82	18.51	16.66	15.14	13.88
1	178.1	209.3	41.86	34.88	29.90	26.16	23.25	20.93	19.02	17.44
1¼	219.7	247.7	49.54	41.28	35.39	30.96	27.52	24.77	22.51	20.64
¾	130.7	141.4	28.28	23.57	20.20	17.67	15.71	14.14	12.85	11.78
1	171.9	177.4	35.48	29.57	25.34	22.17	19.71	17.74	16.12	14.78
1¼	211.9	207.8	41.56	34.63	29.68	25.97	23.09	20.78	18.89	17.31
¾	144.7	186.0	37.20	31.00	26.57	23.25	20.66	18.60	16.91	15.50
1	190.6	234.6	46.92	39.10	33.51	29.32	26.06	23.46	21.32	19.55
1¼	235.3	277.9	55.58	46.31	39.70	34.74	30.88	27.79	25.26	23.16

tions, Stresses, Buckling and Deflection of Wooden Beams

ns and Fiber-Stresses. The cross-sections of wooden beams are variably SQUARE or RECTANGULAR, and those shapes only are considered llowing rules and formulas. Beams should have such a cross-section, maximum fiber-stress due to transverse bending, the maximum horiear and the compression across the grain at the end-bearings, do not e AVERAGE ALLOWABLE UNIT STRESSES as set forth in Table XVI, page

ng. Beams should be braced laterally to prevent BUCKLING when of length to breadth exceeds twenty, or designed with a reduced fiber-n that allowable, where this ratio is exceeded. The *PERCENTAGE OF* N should be as *follows:*

Ratio of length to width......	20 to 30	30 to 40	40 to 50
Percentage of reduction......	25	34	42

Deflection. It is also important that beams carry the loads without
ING beyond a limit fixed by the use to which the structure is applied;
is generally taken at 1/30 of an inch per foot of span for plastered ceiling

3. Constants and Coefficients for Beams

Value of the Constant, A. The letter *A* in the following formu
(16), denotes the SAFE LOAD for a UNIT BEAM, 1 in square in section a
span, loaded at the middle of the span. This is also one-eighteen
ALLOWABLE FIBER-STRESS in pounds per square inch. (See Table I,
557.) The following are the values of *A*, obtained by dividing by ei
RECOMMENDED UNIT STRESSES for TRANSVERSE BENDING, and those gi
building laws of New York, Chicago, Baltimore, Boston, Cincinnat
District of Columbia and in the recommendations of the Board of Fi
writers.

Table II.* Coefficients Recommended for Iron, Steel and Woo
Beams. Values of A

Materials	New York†	Chicago	Baltimore	Boston
Cast iron........	167	167	167	167
Wrought iron.....	667	667	667	667
Steel............	889	889	889	889
Yellow pine......	67	72	100	83
White pine.......	44	44	56	56
Spruce...........	44	44	75	56
Hemlock..........	33	33	....	....
Chestnut.........	44	....	....	....
Oak..............	56	67	83	56

* For safe allowable working unit stresses for other woods, see Table
647. From these values, *A* may be determined by dividing them by eigh
Table XVII, page 648, for other stresses for woods, taken from various bui
See Tables XVIII and XIX, pages 650 and 651, for the ultimate strength
woods.

† New York, Cincinnati, District of Columbia and the Board of F
writers use identical values.

Table III. Coefficients Recommended for Stone ‡ and Concrete
Values of A

Materials	Values of *A*	Materials
Granite..................	10	Bluestone..................
Limestone................	8	Slate......................
Marble...................	7	Concrete 1 : 2 : 4.........
Sandstone................	6	Concrete 1 : 2 : 5.........

‡ Values of *A* for STONE BEAMS as recommended are identical with those o
ing Laws of New York and the Board of Fire Underwriters. For tempo
tures the above recommended values of *A* may be increased 30%.

4. Flexural Strength of Wooden Beams

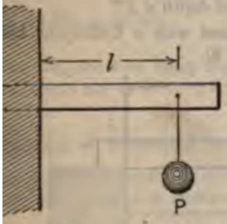
on-Modulus. For beams with a rectangular cross-section, the formulas ngth can be simplified by substituting for the SECTION-MODULUS its bd^2, where b is the breadth and d the depth of the section.

tuting this value in the general formulas for beams with rectangular tions and of any material, the following formulas result:

Fixed at One End and Loaded at the Other (Fig. 5).

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times \text{length in feet}} \qquad (4)$$

$$\text{Breadth, in inches} = \frac{4 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A^*} \qquad (5)$$



5. Cantilever Beam. Load near Free End

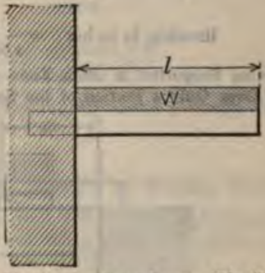


Fig. 6. Cantilever Beam. Distributed Load over Entire Span

s Fixed at One End and Loaded with a Uniformly Distributed Load

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{2 \times \text{length in feet}} \qquad (6)$$

$$\text{Breadth, in inches} = \frac{2 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A^*} \qquad (7)$$

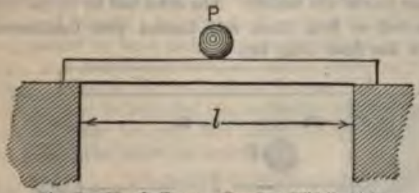


Fig. 7. Simple Beam. Load at Middle of Span

Supported at Both Ends and Loaded at the Middle (Fig. 7).

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{\text{span in feet}} \qquad (8)$$

$$\text{Breadth, in inches} = \frac{\text{span in feet} \times \text{load}}{\text{square of depth} \times A^*} \qquad (9)$$

* *For values of A, see Tables II and III.*

Beams Supported at Both Ends and Loaded with a Uniformly Load Over Entire Span (Fig. 8).

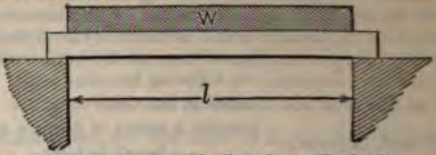


Fig. 8. Simple Beam. Distributed over Entire Span

$$\text{Safe load, in pounds} = \frac{2 \times \text{breadth} \times \text{square of depth} \times A^*}{\text{span in feet}}$$

or

$$\text{Breadth, in inches} = \frac{\text{span in feet} \times \text{load}}{2 \times \text{square of depth} \times A^*}$$

Beams Supported at Both Ends and Loaded with a Uniformly Load Over Only a Portion of the Span (Fig. 9).

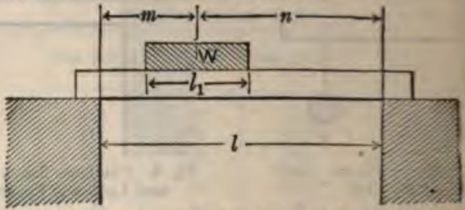


Fig. 9. Simple Beam. Distributed Load over Part of Span

In this case the dimensions of the beam required to carry the accurately determined only by computing the MAXIMUM BENDING explained in Chapter IX, and substituting the value thus found in following. If, however, the length l_1 is very short in comparison with load may be considered as CONCENTRATED at the middle of the span breadth of the beam may be found by Formula (9). Formula (13) the load is at one side of the middle. The error will be on the safe

Beams Supported at Both Ends and Loaded with Concentrated at the Middle of the Span (Fig. 10).

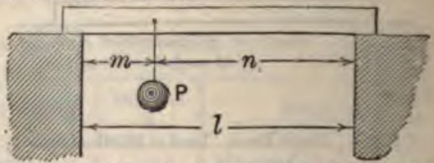


Fig. 10. Simple Beam. Concentrated Load at Any Point

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times \text{span} \times A^*}{4 \times m \times n}$$

or

$$\text{Breadth, in inches} = \frac{4 \times \text{load} \times m \times n}{\text{square of depth} \times \text{span} \times A^*}$$

* For values of A, see Tables II and III.

$$\left.\begin{array}{l}\text{afe load, } P\text{, in pounds}\\ \text{at each point}\end{array}\right\} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times m} \qquad (14)$$

$$\text{readth, in inches} = \frac{4 \times \text{load at one point} \times m}{\text{square of depth} \times A^*} \qquad (15)$$

a the last two cases the lengths denoted by *m* and *n* should be in spans are in feet.

ation of Formulas for Flexural Strength of Wooden Beams

1. What load, 6 ft out from the wall, will an 8 by 14-in long-leaf beam, securely fastened at one end into a brick wall, sustain with

The safe load in pounds (Formula 4) $= \dfrac{8 \times 196 \times 67}{4 \times 6} = 4\,377$ lb

2. It is desired to suspend two loads of 10 000 lb each, 4 ft from f an oak beam, 20 ft long. What should be the size of the beam?

Let the depth of the beam be assumed to be 16 in. Then (For-

$$\text{The breadth} = \frac{4 \times 10\,000 \times 4}{256 \times 67} = 9.3 \text{ in, nearly}$$

therefore, should be 10 by 16 in in cross-section.

ith Several Loads. It is required, next, to determine the size of a h is supported at both ends, and which will safely support several ed loads, or a distributed load and one or more concentrated loads. t method of finding the least size of a beam that will safely support

Table IV. Factors for Equivalent Distributed Loads

	Position of load	Fac
For a concentrated load	Applied at middle of span	Multiply
For a concentrated load	Applied at one-third the span	Multiply
For a concentrated load	Applied at one-fourth the span	Multiply
For a concentrated load	Applied at one-fifth the span	Multiply
For a concentrated load	Applied at one-sixth the span	Multiply
For a concentrated load	Applied at one-seventh the span	Multiply
For a concentrated load	Applied at one-eighth the span	Multiply
For a concentrated load	Applied at one-ninth the span	Multiply
For a concentrated load	Applied at one-tenth the span	Multiply

(See, also, Chapter XV, Safe Loads for Steel Beams, page 566.)

Thus, a concentrated load of 900 lb, applied at one-sixth the span support, will result in the same maximum bending moment as a d load of 900 × $1\frac{1}{9}$, or 1 000 lb.

The above method for finding the size of a beam for a combination loads gives a larger beam than the correct method, by Formula (reason that the maximum bendin will not be equal to the sum dividual bending moments. He there are several heavy loads t ported, it is economical to co maximum bending moment by th METHOD explained in Chapter 329.

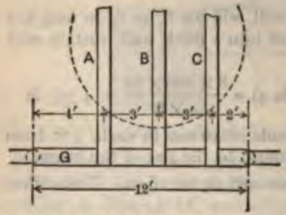


Fig. 12. Girder with Three Concentrated Loads

Example 3. The girder *G*, Fi ports the rafters of a flat roof three heavy beams, *A*, *B* and *C*, above the roof and supporting a filled with water. The timber long-leaf yellow pine. The weight of the roof and allowance for sno lb. Each of the beams *A*, *B* and *C*, impose a load on the girder, weight of the tank and its contents, of 3 000 lb. What should be the girder?

Solution. The roof-load may be considered to be uniformly d The load from beam, *A*, is applied at one-third the span from one end from *B*, five-twelfths the span from the other end; and the load fro sixth the span. The fraction five-twelfths is the mean of one-half and hence the load from *B* should be multiplied by 1.89. Multiplyin centrated loads by their proper factors, the equivalent distributed loa to be as follows:

Roof-load, distributed,	=	7 500
Load from *A*, 3 000 × 1.78	=	5 340
Load from *B*, 3 000 × 1.89	=	5 670
Load from C, 3 000 × $1\frac{1}{9}$	=	3 333
Equivalent distributed load	=	21 843 lb

compression, and add the two breadths together.

4. A spruce tie-beam, 10 ft long between joints, sustains a ceiling-oo lb and a direct tensile stress of 40 000 lb. What should be the of the beam?

. As a ceiling-load is uniformly distributed, the size of the beam is d by Formula (11), page 630. Assuming the depth to be 10 in

$$\text{The breadth} = \frac{10 \times 2\,000}{2 \times 100 \times 39}, \quad \text{or} \quad 2\tfrac{1}{2} \text{ in, nearly}$$

istance of spruce to tension (see Table XVI, page 647) is 800 lb per 000/800 = 50 sq in, which is equivalent to a 5 by 10-in section. It will erefore, a beam 7½ by 10 in in cross-section to resist both the trans-ss and the direct tension. If the tie-beam is cut in any way so as to e section, except over a support, the dimensions must be increased y.

5. A strut-beam of white pine, 10 ft long, supports a distributed of 6 000 lb, and is also subject to a direct compression of 64 000 lb. uld be the size of the beam?

. Assuming 14 in for the depth, the breadth for the transverse load by Formula (11), page 630

$$\text{The breadth} = \frac{10 \times 6\,000}{2 \times 196 \times 39} = 3.9 \text{ in, nearly}$$

Formula (4), page 450, from which is computed Table IV, page 452, e safe loads for white-pine posts, it is found that a 7½ by 14-in post, will safely carry the compressive stress, 64 000 lb. Hence it will re-½ by 14-in beam to resist the compressive stress, and a 4 by 14-in resist the transverse load. The beam, therefore, should be 12 by 14 -section to resist them both.

Table V. Relative Strengths of Rectangular Beams

Kind of load	Position of load	Strength rat
Beam supported at both ends		
Uniformly distributed	Over entire span	1
Concentrated	At middle of span	1/2
Concentrated	At one-third the span	9/16
Concentrated	At one-fourth the span	2/3
Concentrated	At one-fifth the span	25/32
Concentrated	At one-sixth the span	9/10
Concentrated	At one-seventh the span	49/48
Concentrated	At one-eighth the span	8/7
Concentrated	At one-ninth the span	81/64
Concentrated	At one-tenth the span	25/18
Beam fixed at one end, or cantilever beams		
Uniformly distributed	Over entire span	1/4
Concentrated	At the free end	1/8
Beam supported at one end and fixed at the other end		
Uniformly distributed	Over entire span	1
Concentrated	Near the middle of span	1 [illegible]
Beam fixed at both ends		
Uniformly distributed	Over entire span	1 1/2
Concentrated	At middle of span	1

The Strongest Beam Cut From a Cylindrical Log is one in breadth is to the depth as 5 is to 7, very nearly, and the dimensions of su can be found graphically, as shown in Fig. 13. Any diagonal, as ab and divided into three equal parts by the po d; from these points lines perpendicular to ab and the points e and f connected with a shown.

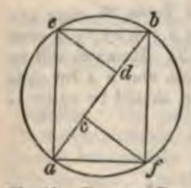


Fig. 13. Strongest Beam of Rectangular Section Cut from Log

Cylindrical Beams. A CYLINDRICAL BE ten-seventeenths as strong as a beam with cross-section, the side of the square being eq diameter of the circular section of the cylindri Hence, to find the safe load for a cylindrical b find the proper load for the correspondin section beam, and divide this load by 1.7.

The Bearing of the Ends of a Beam beyond a certain distance does not strengthen the beam. In genera should have a bearing of 4 in, or if it is very long, 6 in.

The Weight of the Beam Itself. The formulas given for the st beams do not take into account the WEIGHT OF THE BEAMS THEMSE hence the safe loads of the formulas include both the external loa weights of the material in the beams. In small wooden beams, t

of Timber. The weight per cubic foot for different kinds of
ound in the table in Part III, pages 1415 to 1422, giving the
us Substances.

es for Strength and Stiffness of Wooden Beams

o XV for the Strength and Stiffness of Wooden Beams
es 638 to 646, for BEAMS ONE INCH IN BREADTH. To find the
other breadth, multiply the proper tabular value by the breadth
ches. To obtain the required breadth for any load, divide the
nds by the proper tabular value. In heading the tables, prom-
given to the values used for S, and the corresponding values
se who prefer to use for any wood a value different from that
eed only to look up the table based on the value they desire
certain cases and in some cities, the building laws specify 1 300,
pounds as values of S to be used for long-leaf yellow pine;
II, XIV and XV, based on these values, are added.
s very weak in SHEARING in comparison with its strength in
MPRESSION, the safe load a beam of short span can carry is
its resistance to CROSS-BREAKING, but by its RESISTANCE to
the NEUTRAL SURFACE. Wooden beams and joists, therefore,
sioned to safely withstand this SHEARING ACTION. The ratio
to the FLEXURAL STRENGTH is not exactly the same for different
ut for practical use and in the tables it has been assumed to
the WORKING UNIT FIBER-STRESS. As it can be shown * that
span to the depth of a rectangular beam, uniformly loaded,
rtional to its CROSS-BREAKING STRESS and SHEARING WORKING
lar loads are figured for the PERMISSIBLE UNIT FIBER-STRESS,
of the span is twelve or more times the depth of the beam;
lengths, the tabular loads are governed by the SHEAR. To
fe load on beams for a deflection not exceeding 1/360 of the
lues have been placed directly underneath the safe loads for
values are based on the MODULUS OF ELASTICITY, E, given in

OR FLEXURE used in determining the safe uniformly distributed
s is (see Formulas (1), page 333 and (2)′, page 557)

$$M = \frac{SI}{c} = \frac{Sbd^2}{6} = \frac{Wl}{8}$$

$$W = \frac{4\,bd^2S}{3\,l}, \text{ in which } l \text{ is the span in inches}$$

OR SHEAR is

$$W = \frac{4\,bdS_s}{3}$$

aterials of Construction, J. B. Johnson, page 55.

W = total safe load in pounds, uniformly distributed;
b = breadth of the beam in inches;
d = depth of the beam in inches;
l = span, in feet or inches, as noted for the different f
S = unit flexural fiber-stress in pounds per square inch;
$S_s = S/12$ = horizontal unit shearing-stress, in pounds per squa
neutral surface;
E = modulus of elasticity in pounds per square inch.

Example 6. What is the safe, uniformly distributed load, a fiber-stress of 1 200 lb per sq in, for an 8 by 14-in long-leaf supported at both ends, and having a 24-ft clear span?

Solution. From Table XII, the load for a 1-in thickness is 1 090 × 8 = 8 720 lb, the total load for the beam. If the deflec should not be more than 1/360 of the span, the safe load for should not exceed 882 lb. Hence, 882 × 8 = 7 056 lb, is th to be used in this case.

Example 7. What should be the size of a Douglas-fir beam a distributed load of 6 400 lb over a clear span of 18 ft?

Solution. From Table X, it is found that a beam 12 in dee and with an 18-ft span, will support 711 lb. Dividing the l 711, the result is 9 for the breadth of the beam in inches. should be 9 by 12 in, to carry a distributed load of 6 400 lb 18 ft. As the deflection-load of 593 lb can be increased 60% the beam is safe for deflection; if, however, cypress is used, 59 in place of 711, to determine the breadth of the beam. This beam 11 by 12 in.

Different Positions of Loads. To find the safe load, co middle of the span of a given beam, find the safe distributed loa 6, and divide this load by 2. To find the safe load conce s are generally ¼ in scant, up to 4 in in breadth, above which they nt; while in depth they are all generally ½ in less than the nominal afe loads may be obtained by multiplying the safe loads for the g nominal sizes, as given in Tables VII to XV, by the factors given ing table.

I. Conversion Factors for Actual Sizes of Wooden Beams

Cross-sections of beams in inches	Factors	Cross-sections of beams in inches	Factors
1¾×5½	1.47	1¾×11½	1.61
2¾×5½	2.31	2¾×11½	2.53
1¾×6½	1.51	1¾×13½	1.63
2¾×6½	2.51	2¾×13½	2.56
1¾×7½	1.54	1¾×15½	1.65
2¾×7½	2.42	2¾×15½	2.58
1¾×9½	1.58	1¾×17½	1.65
2¾×9½	2.48	2¾×17½	2.60

8. What is the safe load for a 2¾ by 13½-in spruce beam, with n?

From Table VIII, the safe load for a 1 by 14-in beam is 847 lb. this by 2.56, we have 2 178 lb as the safe distributed load for a y 13½ in in cross-section. For a full 3 by 14-in cross-section, the ould be 2 541 lb.

eams. The above formulas may be used for rectangular stone n the proper coefficients, recommended in Table III, page 628, are Sandstone beams should never be subjected to any heavy loads and intels should be relieved by steel beams or by brick arches over them them.

e Beams are generally reinforced with steel rods, but when used nforcement, the coefficient, *A*, given in Table III, is recommended.

Tables VII to XV. The safe loads given in Tables VII to XV for the fiber-stresses indicated; but for greater convenience in using each figure in the units-place of each value may be made a cipher, ure in the tens-place may be increased by one when the unit-figure ater. Thus, 505 would be 500, 506 would be 510, etc.

Table VII. Safe Distributed Loads in Pounds for Rectangular Wooden
Recommended for Hemlock. Maximum Fiber-Stress, S = 600 lb
per sq in. E = 900 000 lb per sq in. A = 33

Span in feet	The first horizontal line gives the depth of the beam in inches The loads are for beams one inch wide and supported at both					
	6	8	10	12	14	16
6	400	533	666	800	933	1 066
7	343	533	666	800	933	1 066
8	300	533	666	800	933	1 066
9	266	474	666	800	933	1 066
10	240 240	427	666	800	933	1 066
11	218 199	388	605	800	933	1 066
12	200 166	356	555	800	933	1 066
13	185 143	328	513	738	933	1 066
14	171 122	305 291	477	686	933	1 066
15	160 107	285 253	445	640	871	1 066
16	150 94	267 222	417	600	817	1 066
17	....	251 197	392 384	565	762	1 003
18	....	237 175	371 343	534	726	948
19	....	225 157	351 308	505	688	898
20	....	213 142	333 277	480 480	653	854
21	....	....	317 252	462 435	623	813
22	....	....	303 229	436 397	594	776
23	....	....	290 211	417 363	568	742
24	....	....	278 193	400 334	545 529	712
25	....	....	....	384 308	523 488	683
26	....	....	....	369 284	503 452	657
27	....	....	....	356 264	482 418	633 625
28	....	....	....	343 245	467 389	609 582
29	....	....	....	....	451 353	589 542
30	....	....	....	....	436 339	569 505

The loads above the heavy black zigzag lines are calculated for resistan
Where two loads are given, the upper load is calculated for strength an
load for a deflection not to exceed 1/360 of the span.

Safe Distributed Loads in Pounds for Rectangular Wooden Beams
led for White Pine, Spruce and Eastern Fir. Maximum Fiber-
S = 700 lb per sq in. E^* = 1 000 000 lb per sq in. A = 39

The first horizontal line gives the depth of the beam in inches
The loads are for beams one inch wide and supported at both ends

	8	10	12	14	16	18
67	622	777	933	1 089	1 244	1 400
00	622	777	933	1 089	1 244	1 400
50	622	777	933	1 089	1 244	1 400
11	552	777	933	1 089	1 244	1 400
80 67	497	777	933	1 089	1 244	1 400
55 21	453	707	933	1 089	1 244	1 400
33 85	415	648	933	1 089	1 244	1 400
16 58	383 374	598	861	1 089	1 244	1 400
00 36	356 323	556	800	1 089	1 244	1 400
37 19	332 281	518	747	1 016	1 244	1 400
75 04	311 247	486 482	700	952	1 244	1 400
...	293 219	458 427	660	897	1 172	1 400
...	276 195	433 381	623	847	1 107	1 400
...	262 175	410 342	590	802	1 048	1 326
...	....	389 308	560 534	762	996	1 260
...	....	370 280	534 484	726	948	1 200
...	....	354 255	509 441	692	906	1 144
...	....	338 234	487 403	662 641	866	1 096
...	....	324 215	468 371	635 588	830	1 050
...	....	....	448 342	610 542	796	1 008
...	....	....	430 316	586 502	766 750	970
...	....	....	415 293	565 465	738 695	934
...	....	....	400 272	544 432	711 646	900
...	....	....	....	526 403	687 602	868 856
...	....	....	....	508 377	664 562	840 800

, dry spruce and Eastern fir, E = 1 200 000 could safely be used, making loads those given in Table XI. See, also, foot-note with Table VII.

Table IX. Safe Distributed Loads in Pounds for Rectangular Wooden
Recommended for California Red Wood and Cedar. Maximum
Stress, $S = 750$ lb per sq in. $E = 700\ 000$ lb per sq in. $A =$

Span in feet	The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both					
	6	8	10	12	14	16
6	500	667	833	1 000	1 167	1 333
7	428 382	667	833	1 000	1 167	1 333
8	375 292	667	833	1 000	1 167	1 333
9	333 231	592 547	833	1 000	1 167	1 333
10	300 187	533 443	833	1 000	1 167	1 333
11	274 155	485 366	757 714	1 000	1 167	1 333
12	250 130	445 307	641 600	1 000	1 167	1 333
13	231 110	410 262	641 512	923 885	1 167	1 333
14	214 95	382 226	595 441	857 763	1 167	1 333
15	....	356 197	556 384	800 665	1 088 1 060	1 333
16	....	333 173	521 337	750 584	1 020 929	1 333
17	....	....	491 299	706 518	961 822	1 254 1 223
18	....	....	463 267	667 462	908 733	1 184 1 090
19	....	....	439 240	632 414	860 658	1 122 982
20	....	....	....	600 374	816 594	1 066 886
21	....	....	....	572 339	778 526	1 016 803
22	....	....	....	547 309	742 491	970 732
23	....	....	....	522 282	710 448	928 670
24	....	....	....	500 260	681 412	890 616
25	....	....	....	480 239	653 380	854 567
26	....	....	....	463 221	628 351	821 525
27	....	....	....	444 205	605 326	791 487
28	....	....	....	428 190	583 203	762 452
29	....	....	....	....	563 282	736 421
30	....	....	....	....	544 264	712 [illegible]

See foot-note with Table VII.

. Safe Distributed Loads in Pounds for Rectangular Wooden Beams
ommended for Douglas Fir, Norway Pine, Cypress and Chestnut.
n Fiber-Stress, $S = 800$ lb per sq in. $E = 900\ 000$ lb per sq in. $A = 44$

The first horizontal line gives the depth of the beam in inches
The loads are for beams one inch wide and supported at both ends

6	8	10	12	14	16	18
533	711	889	1 066	1 244	1 422	1 600
457	711	889	1 066	1 244	1 422	1 600
400 375	711	889	1 066	1 244	1 422	1 600
356 296	632	889	1 066	1 244	1 422	1 600
320 240	569 569	889	1 066	1 244	1 422	1 600
291 199	517 470	809	1 066	1 244	1 422	1 600
267 166	474 395	742	1 066	1 244	1 422	1 600
246 142	438 337	684 658	985	1 244	1 422	1 600
229 122	407 291	635 567	914	1 244	1 422	1 600
214 107	379 253	593 494	854 854	1 161	1 422	1 600
200 94	356 222	556 432	800 750	1 089	1 422	1 600
....	335 197	524 384	754 665	1 025	1 339	1 600
....	316 175	494 343	711 593	968 914	1 264	1 600
....	300 157	468 308	674 532	917 846	1 198	1 517
....	284 142	445 277	640 480	871 752	1 138 1 138	1 441
....	....	423 252	609 435	830 692	1 084 1 032	1 372
....	....	404 229	582 397	792 630	1 035 941	1 309
....	....	387 211	557 363	758 577	990 860	1 253 1 225
....	....	371 193	534 334	726 529	949 790	1 200 1 126
....	....	....	512 308	697 488	911 728	1 152 1 037
....	....	....	492 284	670 452	876 675	1 108 960
....	....	....	474 264	646 418	843 625	1 068 890
....	....	....	457 245	622 389	813 582	1 029 827
....	....	....	....	601 353	785 542	993 770
....	....	....	....	581 339	759 506	960 720

foot-note with Table VII. For safe deflection-loads, for Douglas fir, add
rway pine, 20%, to the above values.

Table XI. Safe Distributed Loads in Pounds for Rectangular Wooden
Recommended for Short-Leaf Yellow Pine. Maximum Fiber-Stress, S
lb per sq in. E = 1 200 000 lb per sq in. A = 55.6

Span in feet	The first horizontal line gives the depth of the beam in The loads are for beams one inch wide and supported at both				
	6	8	10	12	14	16
6	667	889	1 111	1 333	1 556	1 778
7	571	889	1 111	1 333	1 556	1 778
8	500 500	889	1 111	1 333	1 556	1 778
9	444 395	790	1 111	1 333	1 556	1 778
10	400 320	711	1 111	1 333	1 556	1 778
11	364 265	647 628	1 010	1 333	1 556	1 778
12	333 222	593 527	926	1 333	1 556	1 778
13	308 190	547 449	855	1 231	1 556	1 778
14	286 163	508 388	794 757	1 143	1 556	1 778
15	267 143	474 337	741 659	1 067	1 452	1 778
16	250 125	445 296	695 578	1 000 1 000	1 361	1 778
17	....	419 263	654 512	942 886	1 281	1 674
18	....	395 234	618 457	890 790	1 210	1 581
19	....	374 210	585 410	843 710	1 146 1 126	1 498
20	....	356 190	556 370	800 641	1 088 1 016	1 423
21	....	....	528 336	762 581	1 037 922	1 355
22	....	....	505 306	727 529	990 841	1 293 1 254
23	....	....	483 281	696 484	947 770	1 237 1 147
24	....	....	463 258	667 445	908 706	1 186 1 053
25	....	....	....	640 410	871 650	1 138 972
26	....	....	....	615 380	838 602	1 094 900
27	....	....	....	593 352	807 558	1 054 834
28	....	....	....	572 327	778 518	1 016 776
29	....	....	....	....	751 484	982 725
30	....	....	....	....	726 452	949 674

See foot-note with Table VII.

XII. Safe Distributed Loads in Pounds for Rectangular Wooden Beams mmended for White Oak and Long-Leaf Yellow Pine.* Maximum Fiber-Stress, $S = 1\ 200$ lb per sq in. $E = 1\ 500\ 000$ lb per sq in. $A = 66.7$

The first horizontal line gives the depth of the beam in inches The loads are for beams one inch wide and supported at both ends						
6	8	10	12	14	16	18
800	1 067	1 333	1 600	1 867	2 133	2 400
686	1 067	1 333	1 600	1 867	2 133	2 400
600	1 067	1 333	1 600	1 867	2 133	2 400
533 495	949	1 333	1 600	1 867	2 133	2 400
480 400	854	1 333	1 600	1 867	2 133	2 400
437 332	776	1 212	1 600	1 867	2 133	2 400
400 278	711 658	1 111	1 600	1 867	2 133	2 400
369 247	656 561	1 026	1 477	1 867	2 133	2 400
343 204	610 485	953 946	1 371	1 867	2 133	2 400
320 179	569 422	890 824	1 280	1 741	2 133	2 400
300 156	533 371	834 724	1 200	1 633	2 133	2 400
....	502 329	785 642	1 130 1 108	1 537	2 009	2 400
....	474 293	741 572	1 067 990	1 452	1 898	2 400
....	449 263	702 513	1 010 886	1 375	1 795	2 274
....	426 237	666 462	960 802	1 306 1 272	1 708	2 160
....	....	634 420	914 726	1 245 1 154	1 626	2 057
....	....	606 383	872 662	1 188 1 051	1 552	1 968
....	....	579 351	835 605	1 136 962	1 484 1 435	1 878
....	....	556 322	800 557	1 090 882	1 423 1 318	1 800
....	....	....	768 513	1 045 813	1 366 1 215	1 728 1 727
....	....	....	738 473	1 006 753	1 313 1 125	1 662 1 596
....	....	....	711 440	969 698	1 265 1 043	1 600 1 480
....	....	....	686 410	933 648	1 218 970	1 543 1 377
....	....	....	....	902 605	1 178 903	1 489 1 284
....	....	....	....	871 566	1 138 843	1 440 1 200

r safe loads for fiber-stresses *of 1300, 1500 and 1800 lb per sq in,* see Tables XIV *and XV, respectively. See, also, foot-note with* Table VII.

Table XIII. Safe Distributed Loads in Pounds for Rectangular Wooden

Maximum Fiber-Stress, $S =$ 1 300 lb per sq in. $A =$

The first horizontal line gives the depth of the beam in inche
The loads are for beams one inch wide and supported at both

Span in feet	6	8	10	12	14	16
6	867	1 155	1 444	1 733	2 022	2 311
7	743	1 155	1 444	1 733	2 022	2 311
8	650	1 155	1 444	1 733	2 022	2 311
9	567	1 027	1 444	1 733	2 022	2 311
10	520	924	1 444	1 733	2 022	2 311
11	473	840	1 311	1 733	2 022	2 311
12	433	770	1 200	1 733	2 022	2 311
13	400	711	1 111	1 600	2 022	2 311
14	371	660	1 032	1 486	2 022	2 311
15	347	616	963	1 387	1 887	2 311
16	325	578	903	1 300	1 770	2 311
17	....	544	849	1 224	1 664	2 175
18	....	514	802	1 156	1 572	2 054
19	....	487	760	1 095	1 490	1 946
20	....	462	722	1 040	1 415	1 849
21	....	....	688	990	1 348	1 761
22	....	....	657	945	1 286	1 681
23	....	....	628	904	1 230	1 608
24	....	....	602	867	1 179	1 541
25	....	....	....	832	1 132	1 479
26	....	....	....	800	1 088	1 422
27	....	....	....	770	1 048	1 369
28	....	....	....	743	1 011	1 321
29	....	....	....	....	976	1 275
30	....	....	....	....	943	1 232

Loads above the heavy, black zigzag lines are calculated for resistance to

e XIV. Safe Distributed Loads in Pounds for Rectangular Wooden Beams

Maximum Fiber-Stress, $S = 1\ 500$ lb per sq in. $A = 83.3$

an n et	The first horizontal line gives the depth of the beam in inches The loads are for beams one inch wide and supported at both ends						
	6	8	10	12	14	16	18
6	1 000	1 333	1 667	2 000	2 333	2 667	3 000
7	857	1 333	1 667	2 000	2 333	2 667	3 000
8	750	1 333	1 667	2 000	2 333	2 667	3 000
9	667	1 185	1 667	2 000	2 333	2 667	3 000
10	600	1 067	1 667	2 000	2 333	2 667	3 000
11	548	970	1 515	2 000	2 333	2 667	3 000
12	500	890	1 390	2 000	2 333	2 667	3 000
13	462	820	1 282	1 846	2 333	2 667	3 000
14	428	764	1 190	1 714	2 333	2 667	3 000
15	....	712	1 112	1 600	2 178	2 667	3 000
16	....	667	1 042	1 500	2 042	2 667	3 000
17	....	....	982	1 412	1 974	2 510	3 000
18	....	....	926	1 334	1 815	2 370	3 000
19	....	....	878	1 264	1 720	2 246	2 842
20	....	....	....	1 200	1 632	2 133	2 700
21	....	....	....	1 144	1 556	2 032	2 571
22	....	....	....	1 094	1 484	1 940	2 455
23	....	....	....	1 044	1 420	1 856	2 348
24	....	....	....	1 000	1 362	1 780	2 250
5	....	....	....	960	1 306	1 708	2 160
6	....	....	....	926	1 256	1 642	2 076
7	....	....	....	888	1 210	1 582	2 000
8	....	....	....	856	1 166	1 524	1 930
9	....	....	....	....	1 126	1 472	1 862
0	....	....	....	....	1 088	1 422	1 800

ads above the heavy, black zigzag lines are calculated for resistance to shear.

Table XV. Safe Distributed Loads in Pounds for Rectangular Wooden

Maximum Fiber-Stress, $S = 1\,800$ lb per sq in. $A = 100$

Span in feet	The first horizontal line gives the depth of the beam in inch The loads are for beams one inch wide and supported at both					
	6	8	10	12	14	16
6	1 200	1 600	2 000	2 400	2 800	3 200
7	1 030	1 600	2 000	2 400	2 800	3 200
8	900	1 600	2 000	2 400	2 800	3 200
9	800	1 422	2 000	2 400	2 800	3 200
10	720	1 280	2 000	2 400	2 800	3 200
11	655	1 164	1 818	2 400	2 800	3 200
12	600	1 067	1 667	2 400	2 800	3 200
13	554	985	1 539	2 215	2 800	3 200
14	514	914	1 428	2 057	2 800	3 200
15	480	853	1 333	1 920	2 613	3 200
16	450	800	1 250	1 800	2 450	3 200
17	....	753	1 176	1 694	2 306	3 012
18	....	711	1 111	1 600	2 178	2 844
19	....	674	1 053	1 516	2 063	2 695
20	....	640	1 000	1 440	1 960	2 560
21	....	....	....	1 371	1 867	2 438
22	....	....	....	1 309	1 782	2 327
23	....	....	....	1 252	1 704	2 226
24	....	....	....	1 200	1 633	2 133
25	....	....	....	1 152	1 568	2 048
26	....	....	....	1 108	1 508	1 969
27	....	....	....	1 067	1 452	1 896
28	....	....	....	1 029	1 400	1 829
29	....	....	....	....	1 352	1 766
30	....	....	....	....	1 307	1 707

Loads above the heavy, black zigzag lines are calculated for resistance to

8. Working Unit Stresses for Woods

Working Unit Stresses for different woods are given in Table XVI. e compiled and adapted largely from recommended UNIT STRESSES by the Association of Railway Superintendents of Bridges and Build- by the American Railway Engineering Association. (See, also, Table I, .)

XVI. Safe Working * Unit Stresses for Different Woods, in Pounds per Square Inch

f wood	Tension		Compression			Bending †		Shearing ‡	
	With the grain	Across the grain	With the grain: End-bearing	With the grain: Columns § under 15 diams	Across the grain	Extreme fiber-stress	Modulus of elasticity, $E/1\ 000$	With the grain	Across the grain
f safety	Ten	Ten	Five	Five	Four	Six	One	Four	Four
ak.....	1 200	200	1 400	1 000	500	1 200	1 500	200	1 000
ine....	700	50	1 100	800	200	700	1 000	100	500
eaf w pine..	1 200	60	1 400	1 000	350	1 200	1 500	150	1 250
s fir....	800	.....	1 200	900	200	800	1 500	130	900
eaf yel- ine.....	900	50	1 100	800	250	1 000	1 200	130	1 000
ne and way pine	800	50	1 000	750	200	800	1 100	125	750
and rn fir...	800	50	1 200	900	200	700	1 200	100	750
ck......	600	.....	1 100	800	150	600	900	100	600
s.......	600	.....	1 000	750	200	800	900	125	......
....	700	.....	1 100	750	200	700	700	100	400
ut......	850	.....	....	800	250	800	1 000	150	500
nia red l........	700	.....	900	800	150	750	700	75	.....

average ultimate breaking unit stresses may be obtained by multiplying ing unit stresses by the factors of safety at the top of the table-columns.

also, Table I, page 557.

e and some other stresses in the table are higher than those of the building nany cities, larger factors of safety being used in those cities.

larger end-bearing stresses are frequently used for short columns and for ormulas. (See Table I, page 1138.) Lower factors of safety will give higher

orking Unit Stresses for Woods. Taken from Building Laws

Allowable Working Unit Stresses for different woods, taken from ding laws of four cities, are given in Table XVII. The UNIT STRESSES ENSION, COMPRESSION, BENDING and SHEAR.

Table XVII. Working Unit Stresses for Woods, in Pounds per Squa

Kind of stress	Kind of wood	New York*	Chicago	Baltimore§
Tension.....	Yellow pine†.	1 200	1 300	1 800LLYP
	White pine....	800	800	1 000
	Spruce‡......	800	800	1 200
	Hemlock.....	600	600	800
	Chestnut.....	....	....	....
	Oak..........	1 000	1 200	1 500
	Locust.......	....	....	....
	....	....	1 000SLYP	1 200VP
Compression with the grain	Yellow pine†.	1 000	1 110	1 000LLYP
	White pine...	800	700	800
	Spruce‡......	800	700	800
	Hemlock.....	500	500	600
	Chestnut.....	500	....	....
	Oak..........	900	900	1 000
	Locust.......	1 200	....	1 200
	....	....	800SLYP‖	800NCorYP
Compression across the grain	Yellow pine†.	600	250	600LLYP
	White pine...	400	200	400
	Spruce‡......	400	200	400
	Hemlock.....	500	150	500
	Chestnut.....	1 000	....	....
	Oak..........	800	500	600
	Locust.......	1 000	....	1 000
	....	....	250SLYP‖	400NCorVP
Transverse bending	Yellow pine†.	1 200	1 300	1 800LLYP
	White pine...	800	800	1 000
	Spruce‡......	800	800	1 350
	Hemlock.....	600	600	1 000
	Chestnut.....	800	....	....
	Oak..........	1 000	1 200	1 500
	Locust.......	1 200	....	....
	....	....	1 000SLYP‖	....
Shear with the grain	Yellow pine†.	70	130	100LLYP
	White pine...	40	80	85
	Spruce‡......	50	80	90
	Hemlock.....	40	60	75
	Chestnut.....	....	....	....
	Oak..........	100	200	100
	Locust.......	100	....	....
	....	....	120SLYP‖	90VP
Shear across the grain	Yellow pine†.	500	....	500LLYP
	White pine...	250	....	350
	Spruce‡......	320	....	350
	Hemlock.....	275	....	350
	Chestnut.....	150	....	150
	Oak..........	600	....	720
	Locust.......	720	....	....
	....	....	....	400VP

* Stresses named by N. Y., Cincinnati, Dist. of Columbia and Bd of Fir writers about the same. Exceptions: Dist. of Columbia omits hemlock, omi nut in shear across grain and puts spruce and Virginia pine under one caption; nati makes caption of white pine and spruce, with N.Y. white-pine values, and for hemlock, for shear across grain. † Chicago, "Douglas fir and long-leaf yello ‡ Chicago, no values for spruce; spruce-values apply to Norway pine. ‖ Chicag given for short-leaf yellow pine, SLYP. § Baltimore, LLYP is long-leaf yellow or VP, N. Carolina or Virginia pine. ¶ Boston, yellow pine is" yellow pine

of the weights of many specimens tested and agree generally with
alues given in other tables of weights of materials.

Table XVIII.* Average Ultimate Unit Stresses for the Co or Softwoods, in Pounds per Square Inch

Kind of wood	Weight in lb per cu ft, dry	Tension	Compression		Bending (modulus of rupture)	Sh
			With the grain	Across the grain		With the grain
Cedar (white)	19.72 to 20.70	8 000 to 11 400	4 000 to 6 000	700	5 000	400
Cedar (red)	23.66	8 000	4 000 to 7 000	700	5 000	......
Cypress	29.80	4 000 to 6 000	4 000 to 8 000	700 to 800	5 000 to 11 700	500
Hemlock	26.42 to 32.29	6 000 to 8 700	4 000 to 7 420	600 to 700	3 500	350
Pine (white)	25.55	3 000 to 12 000	3 000 to 6 650	700 to 1 000	4 000 to 10 000	225 to 425
Pine (red), (Norway pine)	30.25	5 000 to 13 000	6 000 to 8 000	800 to 1 000	5 000 to 12 300	500
Pine (yellow), (long-leaf)	43.62	6 000 to 13 000	5 000 to 9 500	1 000 to 1 400	7 000 to 14 200	300 to 700
Pine (yellow), (short-leaf)	38.40	5 000 to 10 000	4 000 to 9 000	900 to 1 000	6 000 to 12 400	400 to 700
Douglas fir (Oregon pine)	32.14	9 000 to 14 000	4 880 to 9 800	800 to 1 200	6 500 to 12 100	500 to 600
Redwood (California)	26.23	7 000 to 10 853	3 000 to 4 000	800	4 500	400
Spruce (black)	28.57	5 000 to 19 500	4 000 to 7 850	700	4 000 to 12 000	250 to 400
Spruce (white)	25.25	5 000 to 19 500	4 000 to 7 850	700	4 000 to 12 000	250 to 400

* The higher values of tensile and compressive strengths are for " dry " soned " timber containing from 10 to 15% of water. For safe fiber-str flexure, see Table I, page 557.

XIX.* Average Ultimate Unit Stresses for the Broad-Leaved or Hardwoods, in Pounds per Square Inch

s of wood	Weight in lb per cu ft, dry	Tension	Compression		Bending (modulus of rupture)	Shear	
			With the grain	Across the grain		With the grain	Across the grain
ite).........	40.77	11 000 to 17 000	4 000 to 9 000	1 900	6 300 to 14 200	450 to 1 100	6 280
l)...........	38.96	......	6 800	......	......	......	......
en).........	44.35	......	8 000 to 9 800	1 700	5 100 to 16 000	1 000	6 280
t............	41.00	9 000 to 13 000	5 000	900	5 000	600	1 500
hite).........	45.26	8 000 to 13 000	6 000 to 10 000	1 200	7 300 to 13 600	800	......
....	36.83	15 000 to 18 000	5 600 to 8 500	1 400	6 000 to 12 700	800	5 890
....	46.16 to 52.17	12 800 to 18 000	7 000 to 10 000	2 700 to 3 200	5 400 to 24 300	1 000 to 1 200	6 000 to 7 800
....	45.70	10 500 to 24 800	7 000 to 11 700	......	......	......	7 176
-vitæ	77.12	11 000	8 800	......	......	......	......
hard).........	43.08	8 000 to 10 000	7 000 to 9 940	......	......	......	6 355
white).......	32.84	8 000 to 10 000	6 000 to 7 500	1 700 to 1 900	......	399 to 537	6 355
ny (Central ca)..........	35.00	2 300 to 17 900	6 000	......	10 800	......	......
ite).........	46.35	10 000 to 19 500	4 500 to 11 300	2 000	6 000	750 to 1 000	4 425
estnut)......	53.63	10 000	7 500	......	......	......	......
e)...........	59.21	13 000	9 000	......	......	......	8 480
l and black)	40.75	10 000	4 000 to 8 500	2 300	9 100 to 15 400	1 100	......
(whitewood)	30.00	7 000	4 000 to 5 700	......	......	......	4 418
(white) (but- t)...........	25.46	......	5 000 to 6 800	......	......	......	2 830
(black)......	38.11	12 000	7 500	......	......	......	4 728

higher values of the tensile and compressive strengths are for "dry" or d" timber *containing from 10 to 15% of water. For safe fiber-stresses for* *e Table I, page 557.*

BY

F. H. KINDL

CORRESPONDING MEMBER AMERICAN INSTITUTE OF ARCH

1. Built-Up Wooden Girders

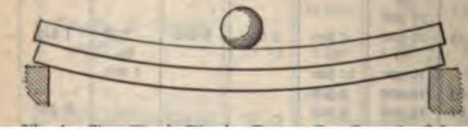
Built-Up Wooden Beams. Wooden beams or girders built
spiked or bolted together side by side, will generally be some
than solid girders of the same dimensions, because the planks
seasoned and freer from check-cracks and other defects. For bea
10 in or less in depth, spikes will usually be sufficient to bind the pl
but for deeper beams, bolts should be used in addition to the spik
the planks from separating and the outer planks from warping or
from the others.

Bolts. Two bolts should be placed at each end of the beam a
feet of its length.

Lengths of Planks. When a beam is built up in this wa
should extend the full length of the beam. In a CONTINUOUS BE
should break joints over the supports. The planks of BUILT-UP
always be set on edge, never flatwise.

Compound Wooden Beams. It is sometimes necessary to
beam for a longer span or greater load than is safe for the deepest
that can be obtained, or for a beam built up of planks. In such cas
WOODEN BEAMS may be used.

Definition. By a COMPOUND WOODEN BEAM OR GIRDER is m
built up by placing two or more single beams over another one,



of having t
SINGLE BEA
depth of t
beams.

Strength
pound Bea

dvocates this as one of the best forms of compound beams, and

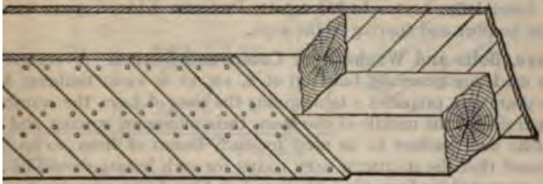
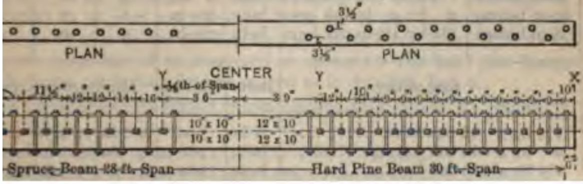


Fig. 2. Simple Form of Compound Wooden Beam

FFICIENCY at about 95% of that of a solid beam of the same depth.
idwell made nine tests of this type of beam. In six of the beams
span to depth was as 12 to 1, and in three of the beams, as 24 to 1.
r beams gave an average EFFICIENCY, without much variation, of
the longer beams an EFFICIENCY of 80.7%.
und that the beams failed by the splitting of the diagonal pieces or
of the nails; "in every case, long before the beam broke, the struts
r the nails were partly drawn out or bent over in the wood, thereby
the component beams to slide on each other." When built with
oards, 1¼ in thick, nailed with tenpenny nails, as in Fig. 2, the
RENGTH of such a beam may be taken at 65% of the strength of



Efficiency of Keyed Beams. Professor Kidwell recomm ordinary purposes an EFFICIENCY of 75% be allowed when oak and of 80% when the keys are of cast iron. The width of an o be twice its height. Numerous small keys closely spaced gave than fewer large keys. In his report, Professor Kidwell gives for the number and spacing of the keys.

Keys, Bolts and Washers for Compound Beams. As com when used, are generally built up of 8, 10, 12 or 14-in timbers some years ago, prepared a table giving the sizes of keys, the nu on each side of the middle of the span, their minimum spacing a the bolts and washers to be used for such beams of from 20 t He noted that the MAXIMUM SAFE LOADS for such beams should loads computed by Formula (10), page 630, for a beam supported and loaded with a uniformly distributed load.

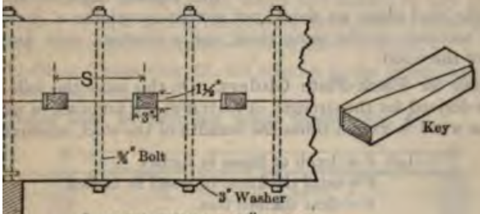
Table I. Keys, Bolts and Washers for Compound, Keyed Wo

Size of beams	Size of keys	Bolts	Washers	Number of k of cent	
				White pine	Spruce
16-in beams	1½ by 3 -in oak keys	¾-in	3 -in	7	8
20-in beams	1½ by 3 -in oak keys	¾-in	3 -in	9	11
24-in beams	2 by 4 -in oak keys	⅞-in	3½-in	8	9
28-in beams	2¼ by 4½-in oak keys	⅞-in	3½-in	9	10

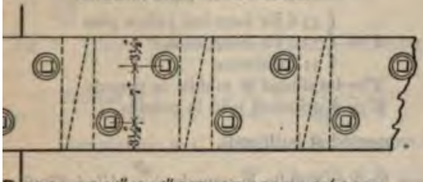
Size of keys	Bolts	Washers	Minimum spa	
1½ by 3 -in oak keys.............	¾-in	3-in	11¼-in	11¼-in

less than the minimum spacing, the safe load should be cor-educed or the thickness of the beam increased.

Uniformly Loaded, the first four or five keys from the end ed for minimum spacing, and the spacing of the remaining keys rd the point *Y*. When the ratio of depth to span is greater than ner key may be a little more than one-eighth the span from the distributed loads. Fig. 3 shows the proper spacing for a 20-in of 28-ft span and for a long-leaf yellow pine beam of 30-ft span; ation below gives the proper spacing of keys for spruce beams of



A ELEVATION OF 20" BEAM



B PLAN OF 14" X 24" SPRUCE BEAM—36′ SPAN

Fig. 4. Details of Keyed and Bolted Wooden Beam

figured from the end of the beam in each case. For other woods spacing should be made as near like these as the fixed condi-mit. Four examples of spacing are given below. The sizes of hers to be used are given in Table I. If the beam is not over bolts may be arranged as for the spruce beam (Fig. 3); if 12 in the bolts should be staggered as shown for the hard-pine beam. beam the bolts might be spaced as in detail *B*, Fig. 4.

eys in inches for spruce beams, commencing at end, for uniformly ads:

beam, 32-ft span, 10, 12, 12, 16, 19, 24, 32
beam, 32-ft span, 10, 11½, 11½, 11½, 12, 12, 12, 13, 15, 18, 24
beam, 36-ft span, 13, 15, 15, 15, 15, 16, 18, 20, 30
beam, 36-ft span, 15, 17, 17, 17, 17, 17, 17, 17, 17, 17

2. Flitched Beams or Flitch-Plate Girders

e Beams (Fig. 5) were at one time much used, but with the of steel it is cheaper and better to use steel beams.

ng explanation and formulas are given, however, for the benefit might have occasion to use a beam of this kind. It has been tice that the thickness *of the wood should be sixteen times the e steel. As the steel is so much* STIFFER *than the wood, we must*

Fig. 5. Flitch-plate Girder

only one-twentieth the
the wooden beam is si
thick as the steel plate, we should put sixteen-twentieths of its s
or, what amounts to the same thing, use a constant only fou
strength of the wood.

Formulas for Flitch-Plate Girders. On this basis the follo
have been derived for the strength of FLITCH-PLATE GIRDERS, in w
ness of the wood is sixteen times the breadth of the steel, approx

Let d = depth of beam in inches
b = total thickness of wood in inches
l = clear span in feet
t = thickness of steel plate in inches
$A' = \begin{cases} 53.6 \text{ for long-leaf yellow pine} \\ 35.2 \text{ for Douglas fir} \\ 31 \text{ for spruce} \end{cases}$
P = total load at middle in pounds
W = distributed load in pounds

Then, for beams supported at both ends,

$$\text{Safe load at middle in pounds} = \frac{d^2}{l}(A'b + 889\,t)$$

$$\text{Safe distributed load in pounds} = \frac{2\,d^2}{l}(A'b + 889\,t)$$

$$\text{For distributed load,} \qquad d = \sqrt{\frac{Wl}{2\,A'b + 1778\,t}}$$

$$\text{For load at middle,} \qquad d = \sqrt{\frac{Pl}{A'b + 889\,t}}$$

The bolts should be ¾ in in diameter, and spaced 2 ft on c
end should have two bolts, as in Fig. 5.

Example. What is the safe load uniformly distributed for a mstances and in some parts of the country it may be cheaper or nient to use a large wooden girder, and truss it, as in Figs. 6, 7, 8 or 9.

f Trussed Girder. For all these forms it is desirable to give the much depth as the conditions allow; as, the deeper the girder, the stresses in the pieces.

ingle-Strut Trussed Girder, we either have two beams, and one runs up between them at the ends, or three beams, and two rods between the beams in the same way. The beams should be in one length for the whole span, if they can be obtained in that length. ite dimensions of the tie-rod, struts and beams, in any given case, etermined by first finding the stresses developed in these pieces, and eas of cross-sections required to resist these stresses.

ingle-Strut Truss (Fig. 6), the stresses in the pieces may be deter- he following formulas:

istributed Load *W* Over the Whole Girder (Fig. 6)

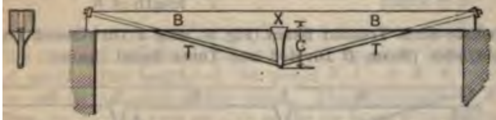


Fig. 6. Trussed Wooden Girder. One Vertical Strut

$$\text{Tension in } T = \frac{W}{2} \times \frac{\text{length of } T}{\text{length of } C} \qquad (5)$$

$$\text{Compression in } C = \tfrac{5}{8} W. \quad \text{(See Note.)} \qquad (6)$$

$$\text{Compression in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } C} \qquad (7)$$

'hen the beam *B* is in one piece, the full length of span. If *B* is r the strut then compression in *C* or tension in $R = \frac{1}{2} W$.

ncentrated Load *P* Over *C* (Fig. 6)

$$\text{Tension in } T = \frac{P}{2} \times \frac{\text{length of } T}{\text{length of } C} \qquad (8)$$

$$\text{Compression in } C = P$$

$$\text{Compression in } B = \frac{P}{2} \times \frac{\text{length of } B}{\text{length of } C} \qquad (9)$$

der Trussed as in (Fig. 7), Under a Distributed Load *W* Over the er

$$\text{Compression in } S = \frac{W}{2} \times \frac{\text{length of } S}{\text{length of } R} \qquad (10)$$

$$\text{Tension in } R = \tfrac{5}{8} W. \quad \text{(See Note.)}$$

$$\text{Tension in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } R} \qquad (11)$$

'hen the beam *B* is in one piece, *the full length of span.* If *B* is r the *strut then compression in C or tension in* $R = \frac{1}{2} W$.

For a Concentrated Load, *P* at the Middle (Fig. 7)

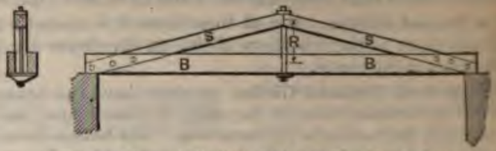


Fig. 7. Trussed Wooden Girder. One Vertical Tie

$$\text{Compression in } S = \frac{P}{2} \times \frac{\text{length of } S}{\text{length of } R}$$

$$\text{Tension in } R = P$$

$$\text{Tension in } B = \frac{P}{2} \times \frac{\text{length of } B}{\text{length of } R}$$

For a Double-Strut Trussed Beam (Fig. 8) with a Distributed Load the Whole Girder (Beam *B* Divided into Three Equal Spans)

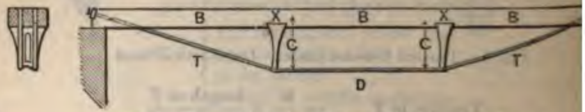


Fig. 8. Trussed Wooden Girder. Two Vertical Struts

$$\text{Tension in } T = \frac{W}{3} \times \frac{\text{length of } T}{\text{length of } C}$$

$$\text{Compression in } C = \frac{W}{3}$$

$$\text{Compression in } B \text{ or tension in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } C}$$

For a Concentrated Load *P* Over Each of the Struts *C* (Fig. 8)

$$\text{Tension in } T = P \times \frac{\text{length of } T}{\text{length of } C}$$

$$\text{Compression in } C = P$$

$$\text{Compression in } B \text{ or tension in } D = P \times \frac{\text{length of } B}{\text{length of } C}$$

For a Girder Trussed as in Fig. 9, and Under a Distributed Load the Whole Girder (Beam *B* Divided into Three Equal Spans)

$$\text{Compression in } S = \frac{W}{3} \times \frac{\text{length of } S}{\text{length of } R}$$

$$\text{Tension in } R = \frac{W}{3}$$

$$\text{Tension in } B \text{ or compression in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } R}$$

ntrated Loads *P* Applied at Joints 2 and 3 (Fig. 9)

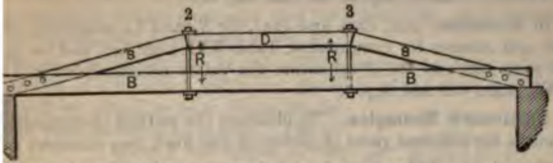


Fig. 9. Trussed Wooden Girder. Two Vertical Ties

$$\text{Compression in } S = P \times \frac{\text{length of } S}{\text{length of } R} \tag{20}$$

$$\text{Tension in } R \quad = P$$

$$\text{Tension in } B \text{ or compression in } D = P \times \frac{\text{length of } B}{\text{length of } R} \tag{21}$$

nstructed as shown in Figs. 8 and 9 should be divided so that the ıe struts *C*, will divide the lengths of the girder into three equal or parts. The lengths of the pieces *T*, *C*, *B*, *R*, *S*, etc. should be THE AXIAL LINES of the pieces. Thus, the length of *R* should be om the CENTER LINE OR AXIS of the tie-beam *B* to the CENTER LINE he strut *D*; and the length of *C* should be measured from the AXIS the AXIS of the strut-beam *B*.

rmining the stresses in the pieces by these formulas, we may com- as of the cross-sections by the following rules:

$$\text{a of cross-section of a short strut} = \frac{\text{compression in strut}}{S_c} \tag{22}$$

for cast iron may be taken at from 13 000 or 14 000 lb per sq in, d as given in Table XVI, page 647.

of the long strut *D* (Fig. 9) should be determined by means of ınd 452 for wooden columns, Chapter XIV.

eters of the tie-rods may be obtained from Table II, page 388.

am *B* (Figs. 8 and 9) when the load is distributed, we must compute area of cross-section as a STRUT (Fig. 8) or a TIE (Fig. 9), and also its cross-section, as a BEAM, required to support its load, and use a section equal to the sum of the two sections thus obtained.

$$\left.\begin{array}{l}\text{a of cross-section of } B \text{ to resist}\\ \text{nsion or compression}\end{array}\right\} = \frac{\text{tension}}{S_t} \text{ or } \frac{\text{compression}}{S_c} \tag{23}$$

s shown in Figs. 6 and 7, with distributed loads,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times l}{4 \times d^2 \times A} \tag{24}$$

es shown in Figs. 8 and 9, with distributed loads,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times l}{6 \times d^2 \times A} \tag{25}$$

Equation (24) and (25) with Equation (11), page 630.)

s the total distributed load in pounds on the girder, and *l* the length ıe section of the beam. When the *loads are concentrated over the* g. 8) *or at the joints R (Fig. 9) then there will be no* TRANSVERSE

Example 1. It is required to design a trussed girder of the
Fig. 6, for a span of 30 ft. The girders are to be 12 ft on cent
carry a floor loaded with 100 lb per sq ft. The girder consists
beams *B*, side by side, and two rods. We can allow the rod *T* to
below the beams *B*, and we will assume that the depth of the
be 12 in; then the length of *C*, measured from the center line of
be 30 in. The length of *B* is 15 ft, and by computation, or by
the length of *T* to be 15 ft 2½ in.

Solution. The total load on the girder equals 100 lb multipli
multiplied by the distance of the girders on centers, or, 100 × 30 ×
From Formula (5),

$$\text{Tension in } T = \frac{36\,000}{2} \times \frac{182\tfrac{1}{2} \text{ in}}{30 \text{ in}} = 109\,500 \text{ lb}$$

or 54 750 lb on each of the two rods. For such a large stress it
the ends of the rods, and allowing 16 000 lb per sq in for steel rod
Table II, Chapter XI, that we must use two 2⅛-in steel rods.

The strut-beam we will make of long-leaf yellow pine. From
find the compressive stress in $B = (36\,000/2) \times (180/30) = 108\,000$
to use three beams side by side, there will be 36 000 lb compressio

To resist the compression there is required an area of 36 000/1
which is equal to 3 by 12 in.

From Formula (24) we find the total breadth required to resist
stress $= \dfrac{36\,000 \times 15}{4 \times 144 \times 67} = 14$ in; or each beam must be 4⅔ by 12
resist the transverse stress, and 3 by 12 in to resist the com
Consequently each beam must be 7⅔ by 12 in in cross-section.

As this would make the girder very wide, 27¼ in, we will us
deep, increasing the depth of the girder 1 in, so that the height
still be 30 in.

The area required to resist the compressive stress will be the
36 in, but as the beam is 14 in deep the breadth will be only 2.5

36 000 ×

It is required to support a floor over a lecture-room 40 ft wide,
russed girders; and as the room above is to be used for electrical
desired to have a truss with very little iron in it. It is decided,
se a truss such as is shown in Fig. 9.

Where the girders rest on the wall, there will be brick pilasters
ection of 6 in, which will make the span of the truss 39 ft, and
will be placed so as to divide the tie-
ee equal spans of 13 ft each. The tie-beam B will consist of two long-leaf yellow pine beams, with the struts S coming between them. There are two rods, instead of one, at RR, coming down on each side of the struts S, and passing through iron castings below the beams B, and forming supports for them. The height of the truss from center to center of timbers must be limited to 18 in. The trusses are 8 ft on centers.

iron Strut
e-rods

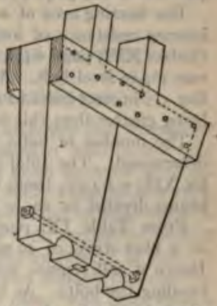


Fig. 11. Wooden Strut for Two Tie-rods

oor-area supported by one girder is 8 by 39 ft, or 312 sq ft. The
to which the floor will be subjected is the weight of the people
for which 75 lb per sq ft is an ample allowance; and the weight
self is about 100 lb per sq ft. This makes the total weight liable
ne girder 31 200 lb.

$$\text{ession in } S, \text{ from Formula (18)} = \frac{W}{3} \times \frac{157 \text{ in}}{18 \text{ in}} = 90\,700 \text{ lb}$$

$$\text{n in one pair of rods} = \frac{W}{3} = 10\,400 \text{ lb}$$

$$\text{n in } B \text{ or compression in } D = \frac{W}{3} \times \frac{156 \text{ in}}{18 \text{ in}} = 90\,130 \text{ lb}$$

ported length of D is greater than that of S, a beam that will
pression in D will be ample for S. We find from Table II, Chap-
it will require a post 10 by 12 in in cross-section to resist the com-
which is 13 ft in length. The tension in each rod is only 5 200 lb;
ds must support a larger washer at the bottom, they are made
ter, not upset. The tension in each of the beams B is 45 065 lb.
by 1 200, the safe unit tensional stress for long-leaf yellow pine
or about 2¾ by 14 in.

readth of the tie-beam to resist the transverse load is found from
Assuming 14 in for the depth of B

$$\text{f } B = \frac{31\,200 \times 13}{6 \times 196 \times 67} = 5.15 \text{ in, or about } 2\tfrac{1}{2} \text{ in for each beam}$$

h of each tie-beam must therefore be 2¾ in + 2½ in = 5¼ in.
beams must be 5 by 14 in in section. The girder, therefore, must
10 by 14-in strut-beams and two 5 by 14-in tie-beams, each 42 ft
in rods may be cut ½ in into the strut-beams, and ½ in into the tie-

one 1½-in steel bolt. As the force to be resisted is 90 700 lb, i
five 1½-in steel bolts to sustain this bearing pressure, the resistanc
being greater than this stress.

The number of bolts required to resist the bending moment r
determined. The total bending moment to be resisted (see pag
ter XII) = 90 700 times the distance, in inches, between the cent
beams divided by 12, or $90\,700 \times {}^{15}\!/_{12} = 113\,375$ in-lb.

From Table IX, page 431, we find that the maximum bend
at a fiber-stress of 20 000 lb per sq in, for a 1½-in steel bolt i
Hence it will require seventeen 1½-in bolts to resist the thrust
bending the bolts. As it is impracticable to put in so many bolt
must be used. For a 2½-in steel bolt, the maximum bendin
30 700 in-lb (Table IX, page 431), and four times this is
hence four 2½-in steel bolts will be sufficient to resist the bendi
also the shearing and bearing-stresses. It will be seen from this
it is much more difficult and expensive to make satisfactory
girders trussed as in Figs. 7 and 9 than it is for the single and
trusses like those shown in Figs. 6 and 8. The latter forms are t
when the conditions will admit of their use.

These four cases of TRUSSED GIRDERS are but special exampl
The stresses in them may also be determined by the methods
Chapter XXVII; and where the divisions of the girder cannot
form, the stresses should be computed by the general methods th

CHAPTER XVIII

IFFNESS AND DEFLECTION OF BEAMS

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General Principles of the Deflection of Beams*

ınd Stiffness. In many structures it is necessary that beams and be not only STRONG enough but STIFF enough; that is, not only :E be prevented, but the beams must not BEND so much as to apy, or to crack the ceiling. Therefore, in many cases, DEFLECTION, absolute STRENGTH, may become the governing consideration in the size of a beam. Unfortunately, there is no method at present the two calculations for STRENGTH and for STIFFNESS in one. A y proportioned for STRENGTH will not bend enough to stress the the ELASTIC LIMIT, but it may in some cases bend more than a or appearances will justify. The distance that a beam bends load is called its DEFLECTION, and its resistance to deflection is 'FNESS.

l Formula for the Maximum Deflection of any beam under a or uniformly distributed load not stressed beyond the ELASTIC

$$\text{inches} = \frac{\text{load in pounds} \times \text{cube of span in inches} \times c}{\text{modulus of elasticity} \times \text{moment of inertia}} \qquad (1)$$

of c are as follows:

n supported at both ends, loaded at the middle......	0.021
n supported at both ends, uniformly loaded.........	0.013
ilever beam, loaded at free end.....................	0.333
ilever beam, uniformly loaded......................	0.125

ı of Beam with Rectangular Section. By making the proper in Formula (1), the following formula for a RECTANGULAR beam T BOTH ENDS and LOADED AT THE MIDDLE may be derived:

$$\text{in inches} = \frac{\text{load in pounds} \times \text{cube of span in feet} \times 1\,728}{4 \times \text{breadth} \times \text{cube of depth} \times E} \qquad (2)$$

of Elasticity. From this formula the value of the MODULUS OF E, for different materials, has been calculated by accurately measural deflection of known beams under given loads applied at the hen substituting these known quantities in Formula (2).

ɔrmula for Deflection. Formula (2) may be simplified someesenting $1\,728/4E$ by $1/F$, which gives the formula

$$\text{Deflection in inches} = \frac{W \times l^3}{b \times d^3 \times F\dagger} \qquad (3)$$

ıUTED LOAD the deflection will be five-eighths of this.

---

n Chapter XVI, formula on page 636 and Table XVI, *page 647.*
ınt *F corresponds to Hatfield's F, in his treatise on "Transverse* Strains."

To Find the Load at the Middle that will cause a GIVEN DEFLECT transpose Formula (2) so that the load becomes the left-hand membe equation. Thus:

$$\text{Load at middle in pounds} = \frac{4 \times \text{breadth} \times \text{cube of depth} \times \text{deflection in in} \times E}{\text{cube of span} \times 1\,728}$$

Limit of Deflection. In order that this formula may be of use in mining the maximum load which may be placed upon a beam, the LIMIT OF DEFLECTION must in some way be fixed. This is generally done by mak certain proportion of the span.

Allowable Deflection of Floor-Beams. Tredgold and other aut state that if a floor-beam deflects more than ONE-FORTIETH OF AN IN EVERY FOOT OF SPAN, it is liable to crack the ceiling on the under side, and this is the limit which is often set for the deflection of beams in fi buildings.

Formulas for Deflection of Floor-Beams. If the length in feet by 40 is substituted for the deflection in inches, Formula (4) becomes

$$\text{Load at the middle} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{square of span in feet}}$$

in which

$$e = \frac{E}{17\,280}$$

Most engineers and architects, however, think that ONE-THIRTIETH INCH PER FOOT OF SPAN, that is, 1/360 of the span, is not too much to the deflection of floor-beams, as a floor is seldom subjected to its full load, and then only for a short time.

Table I. Values of Constants for Stiffness or Deflection on Bea

Material	E	$F = \frac{E}{432}$	$e = \frac{E}{17\,280}$	$e_1 =$
Cast iron	15 000 000	34 722	862	
Wrought iron	26 000 000	60 000	1 500	
Steel	29 000 000	67 130	1 678	
Ash	1 482 000	3 430	87	
California redwood	700 000	1 620	40	
Cedar	700 000	1 620	40	
Chestnut	900 000	2 080	52	
Cypress	900 000	2 080	52	
Douglas fir	1 500 000	3 472	87	
Hemlock	900 000	2 080	52	
Long-leaf yellow pine	1 500 000	3 472	87	
Maple	1 902 000	4 400	110	
Norway pine	1 100 000	2 546	64	
Short-leaf yellow pine	1 200 000	2 777	69	
Spruce	1 200 000	2 777	69	
White oak	1 500 000	3 472	87	
White pine	1 000 000	2 315	58	

E = modulus of elasticity, pounds per square inch;

F = constant for deflection of beam, supported at both ends, and loade middle;

e = constant, allowing a deflection of one-fortieth of an inch per foot of sp

e_1 = constant, allowing a deflection of one-thirtieth of an inch per foot of sp

* See, also, in Chapter XVI, formula on page 656 and Table XVI, page 6

s adopted, the CONSTANT FOR DEFLECTION becomes

$$e_1 = \frac{E}{12\,960}$$

or Stiffness or Deflection of Beams. In either of the above ent that the values used for E, F, e, or e_1, should be derived from s of the same size and quality as those to be used. The values woods given in the following table have been adopted by the areful comparison with the results of numerous tests on large ith values given by different authorities. The editors believe erfectly reliable for first-class merchantable timber.

ulas for Loads, Based Upon the Stiffness of Beams

for Limited Deflections for Rectangular Beams. Knowing caused by a load concentrated at the middle of a beam, and the R DEFLECTIONS, caused by different modes of loading and support- or Cases I to VIII, Figs. 1 to 8, considered under the strength of BEAMS (Chapter XVI), can be easily deduced. These cases, ar- ferent order, are:

For Beams Supported at Both Ends*

middle

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{\text{square of span}} \qquad (6)$$

$$\text{Breadth} = \frac{\text{load} \times \text{square of span}}{\text{cube of depth} \times e_1} \qquad (7)$$

int other than at the middle, *m* and *n* being the segments into n is divided

$$\text{load} = \frac{\text{breadth} \times \text{cube of depth} \times \text{square of span} \times e_1}{16 \times m^2 \times n^2} \qquad (8)$$

$$\text{adth} = \frac{\text{load} \times m^2 \times n^2 \times 16}{\text{cube of depth} \times \text{square of span} \times e_1} \qquad (9)$$

nly distributed

$$\text{Safe load} = \frac{8 \times \text{breadth} \times \text{cube of depth} \times e_1}{5 \times \text{square of span}} \qquad (10)\dagger$$

$$\text{Breadth} = \frac{5 \times \text{load} \times \text{square of span}}{8 \times \text{cube of depth} \times e_1} \qquad (11)$$

m, loaded at the middle‡

$$\text{ad} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{\text{span} \times \text{horizontal distance between supports}} \qquad (12)$$

$$\text{h} = \frac{\text{load} \times \text{span} \times \text{horizontal distance between supports}}{\text{cube of depth} \times e_1} \qquad (13)$$

---

s (6) to (17) the breadth and depth are to be taken in inches, and the n feet. The load is always in pounds.

ven in either of the last two columns of Table I may be used for e or e_1, e degree of stiffness desired, but the values e_1 in the last column are linary conditions.

rmula in Chapter *XVI, page 636.*

Elements of Carpentry, page 65.

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{6 \times \text{square of span}}$$

or,

$$\text{Breadth} = \frac{6 \times \text{load} \times \text{square of span}}{\text{cube of depth} \times e_1}$$

Note. When the span in feet is less than the depth in inc should not be calculated by the formulas for STIFFNESS, but by ZONTAL SHEAR. (See Chapter XVI, page 635.)

3. Relative Stiffness of Beams

Beam supported at both ends and loaded at the middle....
Beam supported at both ends and uniformly loaded........
Beam fixed or restrained at both ends and loaded at the mid
Beam fixed or restrained at both ends and uniformly loaded
Cantilever beam loaded at the free end....................
Cantilever beam uniformly loaded..........................

The Stiffest Rectangular Beam containing a given amoun that in which the ratio of depth to breadth is as 10 is to 6; hen beams, the depth and breadth should be made to approach as n is practicable.

Example 1. What is the greatest distributed load that an 8 pine girder, 20 ft in span, will support, without deflecting at t than one-thirtieth of an inch per foot of span?

Solution. This girder comes under the case of a beam sup ends and loaded with a uniformly distributed load, and hence s lated by Formula (10). Substituting the given dimensions in

$$\text{Safe load} = \frac{8 \times 8 \times 1\,000 \times 77}{5 \times 400} = 2\,464 \text{ lb}$$

Example 2. What should be the dimensions of a long-leaf yel 10 ft in span, to support a concentrated load of 4 250 lb at the will be necessary to use a 4 by 10-in beam. As the span of this : is the same as its depth in inches, it should be tested to see if it requirements for strength also. From Table XII, page 643, it t the safe distributed load for a 1 by 10-in beam, 10 ft in span, is 1 for a 4 by 10-in beam the safe load would be four times as much, The load in this example, however, is applied at the middle; fe distributed load must be divided by 2, which gives 2 666 lb for 1 at the middle. As this is much less than the load to be carried, ne beam should be increased to 4 by 16 in. In general it is not safe ORMULAS FOR STIFFNESS when the span in feet does not exceed the hes.

. What is the largest load that an inclined spruce beam 8 by 12 in ion and 16 ft in length between the supports **will** carry at the sistent with stiffness, the horizontal distance between the supports

Formula (12) is the one to be used in this case. Assuming the ection at one-thirtieth of an inch per foot of span, the value of e_1 posite spruce in the last column of Table I. Making the proper s,

$$\text{Safe load} = \frac{8 \times 1\,728 \times 92}{16 \times 14} = 5\,678 \text{ lb}$$

4. Cylindrical Beams

. The formulas for beams with SQUARE CROSS-SECTIONS may be ams with CIRCULAR CROSS-SECTIONS, if $1.7 \times e$ is substituted for e. ner conditions being equal, a CYLINDRICAL BEAM bends or deflects much as a beam the cross-section of which is the square circumcircular cross-section of the cylindrical beam.

afe Loads for Wooden Beams for a Given Deflection

ables and Formulas. Tables VII to XV, inclusive, pages 638 to he SAFE LOADS FOR BEAMS, give, also, the maximum loads for beams, hat will cause a DEFLECTION not exceeding $\frac{1}{360}$ of the span, that is, ot of span. Where two loads are given for any span or depth the is calculated for STRENGTH and the lower load for DEFLECTION. load is given the calculation is for strength, as the calculation for those particular beams would give an excessive load (Example 2). corresponding load for any thickness, multiply the load given in the breadth of the beam in inches. Suppose, for example, that it o find the greatest distributed load that an 8 by 10-in white-pine in span, will support, without deflecting at the center more than of an inch per foot of span. Referring to Table VIII, page 639, safe loads in pounds for white-pine beams, two values are found 20-ft span, 389 and 308, the latter being the safe load for deflection. oad, therefore, for an 8 by 10-in girder will be eight times, or 464 lb, which agrees with the safe load for the same girder calcuflection by Formula 10, Example 1.

Nominal and Standard Sizes of Wooden Beams

on Factors for Wooden Beams of Standard Size. Table II d for beams that measure less *than the* NOMINAL DIMENSIONS. AMS, *and in many localities floor-joists carried in stock,* are more or less SCANT of the nominal dimensions, and for such joists a redu safe load must be made to correspond to the reduction in size. The SIZES are generally ¼ in scant up to 4 in in breadth, above which they scant; while in depth they are all generally ½ in less than the nom The safe loads may be obtained by multiplying the safe loads a Tables VII to XV, pages 638 to 646, by the FACTORS given in the table:

Table II. Conversion Factors for Beams of Commercial or Stand

Cross-sections of beams in inches	Factors	Cross-sections of beams in inches	Factors
1¾×5½	1.47	1¾×11½	1.61
2¾×5½	2.31	2¾×11½	2.53
1¾×6½	1.51	1¾×13½	1.63
2¾×6½	2.51	2¾×13½	2.56
1¾×7½	1.54	1¾×15½	1.65
2¾×7½	2.42	2¾×15½	2.58
1¾×9½	1.58	1¾×17½	1.65
2¾×9½	2.48	2¾×17½	2.60

Example 4. What is the safe load for a 2¾ by 13½-in spruce span?

Solution. From Table VIII, page 639, the safe load for a 1 by is 847 lb. Multiplying this by 2.56 the product is 2 168 lb, the safe load for a beam 2¾ by 13½ in in cross-section. For a full, "no 3 by 14-in, the safe load would be 2 541 lb.

7. Deflection of Steel Beams

General Formula. The DEFLECTION of any steel beam may means of Formula (1), page 663.

Example 5. It is required to determine the deflection of a beam, 20 ft in span, under its maximum uniformly distribu 9.59 tons.

Solution. The load in pounds = 9.59 tons × 2 000 = 19 180 lb; inches = 20 ft × 12 = 240 in; c, for a beam supported at both en formly loaded, from the values given under Formula (1), is 0.013; 29 000 000 lb per sq in (Table I, page 664); and the moment of the properties of steel I beams, page 355, is 215.8. Substituting in Formula (1), page 663,

$$\text{Deflection in inches} = \frac{19\,180 \times 240^3 \times 0.013}{29\,000\,000 \times 215.8} = 0.551 \text{ in}$$

The allowable deflection is $\frac{1}{30}$ of an inch per foot of span, or $\frac{20}{30}$ =

Coefficients of Deflection. In order to save the time require DEFLECTION-FORMULA, COEFFICIENTS OF DEFLECTION have been wo different spans and are given in Table III.

oefficients of Deflection for Uniformly Distributed Loads*

er-stress, pounds per square inch		Span in feet	Fiber-stress, pounds per square inch		
14 000	12 500		16 000	14 000	12 500
0.014	0.013	26	11.189	9.790	8.741
0.058	0.052	27	12.066	10.558	9.427
0.130	0.116	28	12.977	11.354	10.138
0.232	0.207	29	13.920	12.180	10.875
0.362	0.323	30	14.897	13.034	11.638
0.521	0.466	31	15.906	13.918	12.427
0.710	0.634	32	16.949	14.830	13.241
0.927	0.828	33	18.025	15.772	14.082
1.173	1.047	34	19.134	16.742	14.948
1.448	1.293	35	20.276	17.741	15.841
1.752	1.565	36	21.451	18.770	16.759
2.086	1.862	37	22.659	19.827	17.703
2.448	2.185	38	23.901	20.913	18.672
2.839	2.534	39	25.175	22.028	19.668
3.259	2.909	40	26.483	23.172	20.690
3.708	3.310	41	27.823	24.346	21.737
4.186	3.737	42	29.197	25.548	22.810
4.692	4.190	43	30.604	26.779	23.909
5.228	4.668	44	31.954	28.039	25.034
5.793	5.172	45	33.517	29.328	26.185
6.387	5.703	46	35.023	30.646	27.362
7.010	6.259	47	36.562	31.992	28.565
7.661	6.841	48	38.135	33.368	29.793
8.342	7.448	49	39.741	34.773	31.047
9.052	8.082	50	41.379	36.207	32.328

mission from Pocket Companion, 1915, Carnegie Steel Company.

ection in inches of a section SYMMETRICAL ABOUT THE NEUTRAL section of an **I** beam, channel, zee, etc., divide the coefficient sponding to the given span and fiber-stress by the depth of the To find the deflection in inches of a section NOT SYMMETRICAL RAL AXIS, such as the section of an angle, tee, etc., divide the onding to the given span and fiber-stress by twice the distance er from the neutral axis, obtained from the tables of Chapter X. tion in inches of a section FOR ANY OTHER FIBER-STRESS than given, multiply this fiber-stress by any of the coefficients in given span, and divide by the fiber-stress corresponding to the

quired the deflection of a 10-in 25-lb beam of 10-ft span, under ibuted load of 13 tons, the fiber-stress being taken at 16 000 lb III gives 1.655 as the deflection-coefficient, and dividing this of the beam in inches, the result is 1.655/10 = 0.1655, for the middle. By Formula (1), page 663, the deflection for pan, and load $= \dfrac{26\,000 \times 1\,728\,000 \times 0.013}{29\,000\,000 \times 122.1} = 0.1649$ *in, the*

times the width, the tabular safe loads should be reduced in
the ratios given in the following table in order to insure that th
compression-flanges should not exceed the allowed safe unit str

Length of span	Allowable safe load	Length of span	All
5 × flange-width	Full tabular load	25 × flange-width	71.
10 × flange-width	Full tabular load	30 × flange-width	62.
15 × flange-width	90.6% tabular load	35 × flange-width	53.
20 × flange-width	81.2% tabular load	40 × flange-width	43.

"In addition to this lateral deflection which is induced with
the action of pure bending-stresses, lateral deflection may be
thrust of floor-arches or other loading acting on an axis perp
line of principal bending-stress. The thrust of these arches
neutralized by tie-rods, or the safe carrying capacity of the
computed in accordance with the general formulas of flexure
the combined stresses due to the action of both vertical and h
that is to say, the safe loads should be figured around both t
2-2, and the unit stress computed so as not to exceed 16 000 lb

---

* Adapted by permission from Carnegie's Pocket Companion, 1

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1. General Considerations

s Versus Single-Span Girders. A continuous girder is one three or more supports, as distinguished from a simple girder pon two supports. Continuous girders, except in reinforced-con- ction, and in some types of grillage-foundations, are of rare occur- ding-construction. While in almost every building of importance y to employ girders resting upon piers or columns, placed from 15 t, and while in many cases steel girders could conveniently be ob- would span two and even three of the bays between the supports, tically limited to one-story buildings, because in tall buildings it is uction to have the vertical rather than the horizontal supports Many different opinions are held as to the relative strength s of continuous and non-continuous girders, and different formulas oposed from time to time; but in this chapter the mathematical ill not be given.*

s Girders and Overhanging Girders. In all continuous end-spans (Fig. 2) are somewhat in the condition of a simple one overhanging end, while the other spans are somewhat in of a simple girder with two overhanging ends. At each inter- port there is a negative bending moment, the effect of which he bending moments between the supports.

pporting Forces or Reactions of Continuous Girders

s Girder of Two Equal Spans. Concentrated Load at the ach Span. If a girder of two spans, each equal to l (Fig. 1), be middle of the left span with P lb, and at the middle of the right lb, the reaction at the support R_1 is determined by the formula

$$13\,P - 3\,P_1$$

If $P = P_1$, then each of the end-supports must support $\frac{5}{16} P$ and th support $\frac{22}{16} P$. If the girder is cut so as to make two girders of each, then the end-supports will carry $\frac{1}{2} P$ or $\frac{8}{16} P$, and the mid $\frac{16}{16} P$. Hence, it is seen that the continuous girder of three reactions of the end-supports ished, while the reaction at the support is increased.

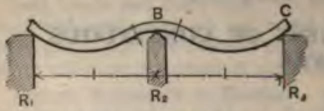

Fig. 1. Continuous Girder of Two Spans

Continuous Girder of Two Uniformly Distributed Load On Span (Fig. 1). Load over each span equals w lb per unit of length be the length of the left span and l_1 the length of the right span. Re left support

$$R_1 = \frac{w}{2}\left[l - \frac{l_1^3 + l^3}{4\,l\,(l + l_1)}\right]$$

Reaction of middle support,

$$R_2 = w\,(l + l_1) - R_1 - R_3$$

Reaction of right support

$$R_3 = \frac{w}{2}\left[l_1 - \frac{l_1^3 + l^3}{4\,l_1\,(l + l_1)}\right]$$

When both spans are equal to l, the reaction of each end-support is ⅜ of the middle support $\frac{5}{4}\,wl$; hence the girder, by being continuous, red reactions of the end-supports, and increases that of the middle suppor 25%.

Continuous Girder of Three Equal Spans. Concentrated Load of P P the Middle of Each Span (Fig. 2).

Reaction of either abutment

$$R_1 = R_4 = \tfrac{7}{20}\,P$$

Reaction of either middle support

$$R_2 = R_3 = \tfrac{23}{20}\,P$$

or the reactions of the two end-supports are $\frac{3}{10}$ less, and those two middle supports $\frac{3}{20}$ greater than they would have been had three separate girders of the same cross-section been used, instead of one continuous girder.

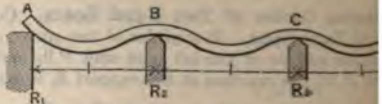

Fig. 2. Continuous Girder of Three Spans

Continuous Girder of Three Equal Spans. Uniformly Distributed Lo Each Span (Fig. 2). The load per unit of length is w lb.

Reaction of either end-support

$$R_1 = R_4 = \tfrac{2}{5}\,wl$$

Reaction of either middle support

$$R_2 = R_3 = \tfrac{11}{10}\,wl$$

Hence the reactions of the end-supports are ⅕ less, and of the middle s $\frac{1}{10}$ more, than if the girder were not continuous.

section.

ous Girders of Two Spans. When a beam is at the point of breaking , the FLEXURE-FORMULA, $M = SI/c$, is frequently used to calculate a UNIT STRESS developed in the beam; and when the beam has a rec- ross-section the formula takes the form (see page 635)

$$\text{um bending ent} = \frac{\text{Modulus of rupture}^* \times \text{breadth} \times \text{square of depth}}{6} \qquad (11)$$

hat the beam may carry its load with perfect safety, the breaking-load divided by a proper FACTOR OF SAFETY. Hence, if the MAXIMUM MOMENT of a beam can be found under any conditions, the required s of the beam can easily be determined from Formula (11). (See page 557, for the safe values of the FIBER-STRESSES.) The greatest oment for a continuous girder of two spans is almost always over the pport, and is a MINUS BENDING MOMENT, if the plus sign is given to num bending moments between the supports. It is the NUMERICAL y, however, that is considered.

ous Girder of Two Spans. Distributed Load over Each Span. The ending moment in a continuous girder of two spans, l and l_1 (Fig. 1), th a uniformly distributed load of w lb per unit of length is over the pport and is

$$\text{Maximum bending moment} = \frac{wl^3 + wl_1^3}{8\,(l + l_1)} \qquad (12)$$

l_1, or both spans are equal,

$$\text{Maximum bending moment} = wl^2/8 \qquad (12a)$$

he same as the maximum bending moment of a beam supported at and uniformly loaded over its whole length. Hence a continuous wo spans uniformly loaded is no stronger as far as flexure is concerned n-continuous.

ous Girder of Two Equal Spans. Concentrated Load at the Middle Span. The greatest bending moment in a continuous girder of two ports, and is determined by the formula

$$\text{Maximum bending moment} = \frac{wl^3 + wl_1^3}{4\,(3\,l + 2\,l_1)}$$

When the three spans are equal, this becomes

$$\text{Maximum bending moment} = wl^2/10$$

or ⅕ less than what it would be were the beam not continuous.

Continuous Girder of Three Equal Spans. Concentrated Load the Middle of Each Span. The greatest bending moment i girder of three equal spans, each of a length l, and each loade with P pounds, is

$$\text{Maximum bending moment} = \tfrac{3}{20}\,Pl$$

or ⅖ less than that of a non-continuous girder.

4. Deflection of Continuous Girders

Continuous Girder of Two Equal Spans. Uniformly Distribu Each Span. The greatest deflection of a continuous girder of loaded with a uniformly distributed load of w lb per unit of len

$$\text{Maximum deflection} = 0.005416\,wl^4/EI$$

in which E is the MODULUS OF ELASTICITY and I the MOMENT OF cross-section of the beam. The greatest deflection of a similar at both ends and uniformly loaded is

$$\text{Maximum deflection} = 0.013020\,wl^4/EI$$

Hence the deflection of the continuous girder is only about ⅖ continuous girder. The greatest deflection of a continuous gird is not at the middle of either span, but between the middle poin one of the abutments. The greatest deflection of a continuou equal spans, loaded at the middle of one span with a load of middle of the other with P_1 lb, is, for the span with the load, P

test deflection in the end-spans $= 0.006884\,wl^4/EI$ (19)

maximum deflection of the continuous girder is only about ½ that ontinuous girder.

us Girder of Three Equal Spans. Concentrated Load *P* at the Middle pan.

atest deflection at the middle span $= \frac{1}{480}\,Pl^3/EI$ (20)
atest deflection at the middle of end-spans $= \frac{11}{960}\,Pl^3/EI$ (21)

maximum deflection of the continuous girder is only $\frac{11}{20}$ of that of ntinuous girder.

s on Reactions, Strength and Stiffness of Continuous Girders

and Reactions of Continuous and Non-Continuous Girders. From ing, some conclusions can be drawn which will be of use in deciding is best in any case to use a CONTINUOUS or a NON-CONTINUOUS From the formulas given for the reactions of the supporting forces erent cases of continuous girders it is seen that the end-supports do s much of the load as they do when the girders are non-continuous. ence is added to the reactions of the other supports. This might be age in a building in which the girders run across the building, and outside ends supported by the side walls and their inside ends by lumns. In this case, by using continuous girders, part of the load aken from the walls and transferred to the piers or columns. But in is kind, the vibration may have to be considered. If the building r factory in which the girders support machines, any vibration in the n of the girder is carried to the side walls if the girder is continuous; on-continuous girders are used, with their ends an inch or so apart, vibration is carried to the side walls from the middle span. In all important construction the supporting forces should be carefully

Strength of Continuous and Non-Continuous Girders. As the STRENGTH of continuous and non-continuous girders of the same on, material and spans, and loaded in the same way, is proportional aximum BENDING MOMENTS, the strength of a continuous girder can ted, from the formula for its MAXIMUM BENDING MOMENT. From the en for these bending moments for the various cases considered, it is the parts of the girder most stressed are those which come over the pports. It is seen, also, that, except in the single case of a girder of uniformly loaded, the strength of a continuous girder is greater of a non-continuous girder. But the gain in strength in some in- not very great, although it is *generally enough to pay for making the* tinuous.

are used they may be made of just the size required for strengt
deflections are less. Where great stiffness is required, theref
beams or girders should be used if possible, as in the case of
(See Example 3, page 679.)

6. Formulas for the Strength and Stiffness of Continu

Girders of Rectangular Cross-Section. For convenience, the
for calculating the strength and stiffness of continuous girders
cross-section are given. The formulas for strength are deduced fr
formula $M = SI/c$, modified for the rectangular section of breadt

$$\text{Bending moment} = \frac{b \times d^2 \times S}{6}$$

in which S is the safe unit fiber-stress. This is eighteen times w
known as the coefficient of strength.*

STRENGTH. **Continuous Girder of Two Equal Spans. Unifor**
Load Over Each Span.

$$\text{Breaking-load}\,\dagger = \frac{2 \times b \times d^2 \times A^*}{l}$$

where b denotes the breadth and d the depth of the girder in in
length of one span, in feet. The values of the constant A are t
values given in Table II, page 628. For long-leaf yellow pine, 20
fir and chestnut, 132; and for spruce and white pine, 117 lb per
ommended for the values of A in these formulas.

Continuous Girder of Two Equal Spans. Concentrated Load a
Each Span.

$$\text{Breaking-load} = \tfrac{4}{3} \times \frac{b \times d^2 \times A}{l}$$

s. **Continuous Girder of Two Equal Spans. Uniformly Distributed Each Span.**

wing formulas give the loads which the beams will support without nore than one-thirtieth of an inch per foot of span.

$$\text{Load on one span} = \frac{b \times d^2 \times e_1}{0.26 \times l^2} \qquad (27)$$

us Girder of Two Equal Spans. Concentrated Load at the Middle of

$$\text{Load on one span} = 16/7 \times \frac{b \times d^2 \times e_1}{l^2} \qquad (28)$$

us Girder of Three Equal Spans. Uniformly Distributed Load Over

$$\text{Load on one span} = \frac{b \times d^3 \times e_1}{0.33 \times l^2} \qquad (29)$$

us Girder of Three Equal Spans. Concentrated Load at the Middle an.

$$\text{Load on one span} = 20/11 \times \frac{b \times d^3 \times e_1}{l^2} \qquad (30)$$

e of the constant e_1 is obtained by dividing the MODULUS OF ELAS- 2 960; and, for the three woods most commonly used as beams, the alues may be taken:

f yellow pine, 116; white pine, 77; spruce, 92; Douglas fir, 116. woods, see Table I, page 664.)

inuous Steel Beams the requisite size may be found by first com- MAXIMUM BENDING MOMENT, by means of Formulas (12) to (15), lecting a beam that has a

$$\text{ECTION-MODULUS} = \frac{3 \times \text{maximum bending moment in ft-lb}}{4\,000} \qquad (31)$$

r the SECTION-MODULI for the different shapes of rolled steel used as given in the tables in Chapter X.

1. What steel beam should be used to support two loads of 16 000 ncentrated at the middle of two spans of 10 ft each, the beam being

Formula (13*a*) gives the maximum bending moment as $\frac{3}{16}$ *Pl*, or . Therefore, from Equation (31), a beam having a section-modulus × 30 000/4 000 or 22.5 should be used. From the Table IV, page ound that a 9-in 30-lb beam has a section modulus of 22.6, and a beam a section modulus of 24.4. Either of these beams will there- , the 10-in beam being the cheaper, however, and also the stiffer.

2. A steel beam continuous over three spans is required to support distributed load of 1 000 lb per lin ft. The two end-spans are 12 ft he middle span 10 ft. What should be the size and the weight of the

The maximum bending moment is found by Formula (14), and is

$$\frac{1\,000 \times 1\,000 + 1\,000 \times 1\,728}{4\,(30 + 24)} = 12\,630$$

The section-modulus, by Equation (31), must equal $3 \times 12\,630/4\,000$ which requires a 7-in 15-lb beam (Table IV, page 355).

If the beam were not continuous an 8-in 18-lb beam would be the 12-ft spans, and a 7-in 15-lb beam for the 10-ft span.

For a beam of two equal spans, loaded uniformly, the strength is the though it were not continuous.

The formulas given for the reactions at the supports, and for the of continuous girders with concentrated loads, were verified by Mr. means of careful experiments on small steel bars. The remaining were verified by comparing them with the formulas of other it was possible to do so. In regard to some of the cases given the never seen any discussion of them in any work on the subject.

7. Continuous Girders in Grillage-Foundations

Grillage-Beams Considered as Inverted Continuous Girders. As the beginning of this chapter, CONTINUOUS GIRDERS, as such, are seldom building-construction, although their employment in grillage-beam

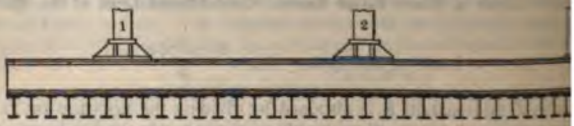

Fig. 3. Continuous Girder in Grillage-foundation

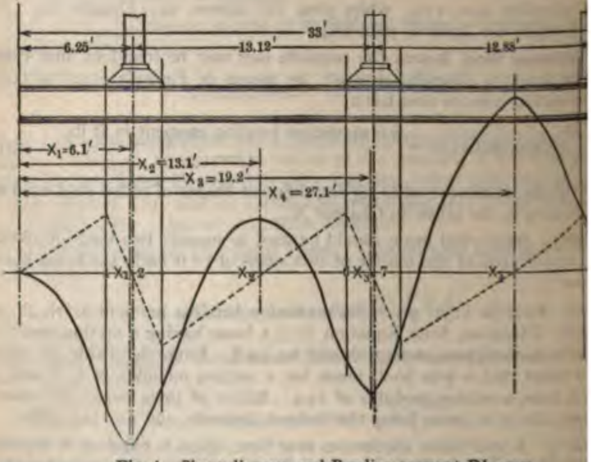

Fig. 4. Shear-diagram and Bending-moment Diagram

quent. Fig. 3 represents a footing consisting of two layers of beams, tribute the load of the three columns above, uniformly over the By inverting the footing the three columns become the supports or the upper layer of beams, a continuous girder, loaded with a uniformly uted load which is the pressure of the lower layer. As in practice the ever equal, and the distance between the columns seldom equal, it to project the continuous girder beyond the most heavily loaded rder to insure a uniform pressure upon the lower layer. Because itations none of the formulas previously deduced can be applied, e principles upon which they are based hold good.

Bending Moment. Since the REACTIONS in this case are the given s it is required first to find the MAXIMUM BENDING MOMENT. From lready been said about continuous girders, it is evident that the ximum bending moment may be under columns 1 or 2, or between s. Since the maximum bending moments are the POINTS OF NO struct the SHEAR-DIAGRAM, find where the shear passes through lculate the bending moments at these points. The maximum bend- is determined, as in examples 1 and 2, in order to determine the DULUS of the girder.

3. The continuous girder under columns 1, 2 and 3 (Fig. 3) is the overhang, to the left of column 1, 6.25 ft; the distance between nd 2, 13.12 ft; between columns 2 and 3, 12.88 ft; and from column nt edge of the girder, .75 ft. The column-loads are as follows: on 55 tons; on column 2, 600 tons; and on column 3, 255 tons.

nn-loads may be considered uniformly distributed over parts of the e bases, which are 3 ft wide under columns 1 and 2 and 18 in wide n 3. The UNIT PRESSURE under column 1, therefore, is $565/3 =$ under column 2, $600/3 = 200$ tons; and under column 3, $255/1.5$ The unit pressure under the continuous girder is

$$(565 + 600 + 255)/33 = 43 \text{ tons}$$

step in the calculation of the girder is the determination of the O SHEAR and the plotting of the SHEAR-DIAGRAM in Fig. 4. It is n the shear-diagram that there are four points of no shear and con- ur points of POSSIBLE MAXIMUM BENDING MOMENT. The first of er column 1, the second between columns 1 and 2, the third under nd the fourth between columns 2 and 3. The bending-moment shown by the solid curved line in Fig. 4. The points of contra- bending moment are the intersections of this line with the hori- f reference.

R-DIAGRAM,* shown by the broken line in Fig. 4, may be constructed

$+ 43$ tons per ft $\times$ $(6.25 - 1.5 = 4.75$ ft$) = + 204.25$ tons

$(+ 43$ tons per ft $\times$ 6.25 ft$) - 565/2$ tons $= + 268.75 - 282.5$

$- 13.75$ tons

s that x_1, the point of no shear, lies between points 1 and 2. To int let y be its distance beyond or to the right of point 1. Then, N FOR NO SHEAR is 43 tons $\times$ $(4.75 \text{ ft} + y \text{ ft}) = 188.3 \times y$, or 204.25 $8.3\,y$, from which $145.3\,y = 204.25$ and $y = 1.4$ ft: hence x_1, the OF NO SHEAR, is 4.75 ft $+ 1.4$ ft, or 6.1 ft from the left end.‡

ND POINT OF NO SHEAR, x_2, is such a distance from the left end that ARD SHEARING-FORCE of 565 tons from column 1 is neutralized by

---

ard forces are here called plus or positive and the downward forces minus

en at point 1, the left edge of base of Column 1, V_2 at point 2, at the axis of

wing computations are carried out to one decimal-place, only, the nearest values being used.

point is 43 tons × (17.9 ft + y ft) = 565 tons + (200 tons × y ft),
= 565 + 200 y, from which 157 y = 204.7 and y = 1.3 ft. Henc
POINT OF NO SHEAR, is 17.9 ft + 1.3 ft = 19.2 ft from the left end

The FOURTH POINT OF NO SHEAR, x_4, is such distance from th
the DOWNWARD SHEARING-FORCE of columns 1 and 2, amountin
or 1165 tons, is neutralized by an equal UPWARD SHEARING-FO
per ft on x_4 ft. Hence x_4 = 1 165/43 = 27.1 ft.

Having found the points of no shear, the BENDING MOMENT
may now be determined.

M at x_1 = 43 tons × 6.1 ft × 6.1/2 ft − 188.3 tons × 1.4 ft × 1.4
= + 615.5 ft-tons

M at x_2 = 43 tons × 13.1 ft × 13.1/2 ft − 565 tons × 6.8 ft =

M at x_3 = 43 tons × 19.2 ft × 19.2/2 ft − 565 tons × 12.9 ft −
× 1.3/2 = + 467 ft-tons

M at x_4 = 43 tons × 27.1 ft × 27.1/2 ft − 565 tons × 20.8 ft − 6
× 7.7 ft = − 582.2 ft-tons

The MAXIMUM BENDING MOMENT therefore is at x_1 and equals 6
1 231 000 ft-lb. Substituting in Formula (31), the SECTION-MO
to be $\frac{3 \times 1\,231\,000}{4\,000}$ = 923.2. The following beams could be
flexure is concerned. For investigations of the resistance of the
buckling or crippling, see Chapter II, pages 182 to 184, and
pages 567 to 569.

Four standard 24-in 110-lb **I** beams, section-modulus of each, 2

Three Bethlehem 30-in 120-lb **I** beams, section-modulus of ea
357)

Three Bethlehem 24-in 140-lb girder-beams, section-modulus
(page 358)

Two Bethlehem 28-in 180-lb girder-beams, section-modulus
(page 358)

The 28-in and 30-in beams are stiffer than the 24-in beams,

CHAPTER XX

IVETED STEEL PLATE AND BOX GIRDERS

BY

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1. General Notes on Plate and Box Girders

f Riveted Girders. Girders built up of plates and angles, as shown in Figs. 1 to 4, are extensively used. This is undoubtedly owing to ity of their construction, to the comparatively low cost of the shapes hey are fabricated and to their adaptability to any arrangement of any span for which girders are usually required. Riveted girders, re seldom made for spans greater than 60 ft and are seldom more in depth. The most common forms of these girders are shown in l 4.

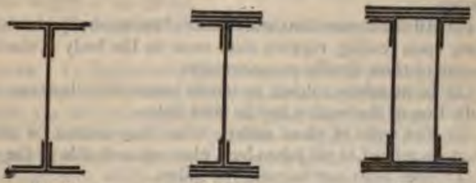

Fig. 2 Fig. 3 Fig. 4

Types of Riveted Girders

lers with a single vertical plate called the WEB are usually called DERS, and those with double or triple webs, BOX GIRDERS. Plate more economical than box girders, and more accessible for painting tion; but box girders are stiffer laterally and should always be used t length of span requires wide flanges. In general, it may be said girders should be used to support floor-beams and floor-arches and ver 12 in thick, and that box girders should be used where a flange-ter than 12 in is required. The girder shown in section in Fig. 1 has lates and should be used only for comparatively light loads and short never to support masonry.

and Web. The term FLANGE, as applied to riveted girders, includes al in the top or bottom parts of the girder, exclusive of the web-plates.* t of a riveted girder is the distance between the centers of gravity of ; but in practice this is usually taken as the DEPTH OF THE WEB-l the word will be so used in this chapter. The top and bottom of angles extend ¼ in beyond the top and bottom of the web-plate. gure in Table IV, page 706.) STIFFENERS are short pieces of angles

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ay be modified, however, as some engineers include one-sixth of the web-area ive flange-area. See, also, *Flange-Area in the examples of this chapter.*

ECONOMIC DEPTH when the amount of material in the flanges in the web, and there are no cover-plates. The rule holds when there are cover-plates.

The Width of the Top Flange should not be less than o distance between lateral supports; or if there are no lateral su less than one-twentieth the span.

Arches Between Girders, or floor-beams riveted to the sides be considered as LATERAL SUPPORTS.

2. Details of Construction of Plate and Box G

General Requirements for Plate and Box Girders. T quirements are those which must be generally satisfied in the d girders.

(1) All the connections and details of the several parts shall be that, upon testing, rupture shall occur in the body of the mem in any of their details or connections.

(2) In members subject to tensile stress full allowance shall reduction of the section by the rivet-holes.

(3) The webs of plate girders, when they cannot be obtaine must be spliced at all joints by a plate on each side of the web.

(4) Tees must not be used for splices.

(5) Stiffeners shall be used at the ends of all girders, whereve centrated loads, and elsewhere when the shearing-stress is g resistance to buckling.

(6) The PITCH, that is, the distance between centers of rive ceed 6 in, nor 16 times the thickness of the thinnest outside pl not be less than 2¼ in for ¾-in rivets, or 2⅝ in for ⅞-in rive line.

(7) The rivets used should be ¾ in in diameter for plates thick, and ⅞ in in diameter for plates of greater thickness.

(8) The distance between the edge of any piece and the ce hole must never be less than 1¼ in.

(9) In PUNCHING plates or other members, the diameter of th

ienever possible, all rivets must be MACHINE-DRIVEN.
e several pieces forming one built member must fit closely together,
riveted, must be free from twists, bends or open joints.
rders 60 ft and less in length seldom require SPLICING, as the plates and
readily be obtained in such lengths. In splicing the top flange,
wo or more thicknesses, no additional COVER-PLATE will be required
int, but the ends should be planed true and butt closely. The rivets
spaced closer near the joint.
e plate covering the bottom flange must be of the same area as the
ed, and of sufficient length to take a number of rivets equal to the
f the cover-plate.

3. Design of Plate and Box Girders

incipal Steps in the Design of Riveted Girders. In designing
irder to sustain with safety a given load, the following steps are neces-

determination of the required flange-area.
determination of the thickness of the web to resist
i) Shearing,
) Buckling. This step also determines whether or not stiffeners are
necessary.
determination of the number and pitch of the rivets.
approximate weight of the girder.
determination of the length of the flange-plates when more than one
for each flange.

e Flange-Area. In determining the FLANGE-AREA of riveted girders,
nary to assume that the bending moments are entirely resisted by the
lower flanges, the web being assumed to resist the shear only. Just
d be included in the flange-area is a question on which engineers differ.
de the flange-plates and angles and one-sixth of the web-area, others
plates and angles only, while others include the flange-plates and only
ntal legs of the angles, the vertical legs being considered as belonging
. In compression-flanges, usually the upper ones, the gross section-
be taken, provided the rivets are machine-driven and fill completely
but in tension-flanges, usually the lower ones, the net area is taken,
e gross area minus the area of the greatest number of rivet-holes in
section, since the stresses of tension are not transmitted through the
re those of compression.
al FORMULA* FOR DETERMINING THE FLANGE-AREA, which applies to
ns of loading is

$$\begin{array}{l}\text{one flange}\\ \text{square inches}\end{array} = \frac{\text{maximum bending moment in foot-tons}}{\text{depth of web in feet} \times \text{safe unit fiber-stress in tons}}$$

$$A = M_{\max}/dS \qquad (1)$$

ay be derived from what is sometimes called the PLATE-GIRDER FORMULA,
d, in which S is the safe unit bending-stress in the flange at the section of
ending moment, A is the area of the cross-section of either flange and d is ap-
the depth of the girder. Of course the units must be the same in both mem-
equation. If the center of moments is taken at the center of gravity of the
of either flange-area and if the area of metal resisting bending is considered
ntrated in the flanges, the depth of each being very small compared to that
r-depth, then SA is the total horizontal stress in either flange, d its lever-
Ad the resisting moment of the cross-section, equal to $M_{\max}$. Hence $A =$

span in feet

or

$$W = 8AdS/l$$

From the result the weight of the girder itself should be subtrac
For the SAFE CONCENTRATED LOAD AT THE MIDDLE OF THE SPA
the result obtained by formula (2) and subtract the weight c
Case IV, page 326.)

(2) **The Thickness of the Web.** The thickness of the wel
by its resistance to VERTICAL SHEARING. Whether or not stiffene
is determined by the resistance of the web to BUCKLING.

(a) **Shearing.** To resist the vertical shear the NET SECTIONA
WEB in square inches must be

$$A = \frac{\text{the maximum vertical shear}}{S}$$

or

$$A = V_{\max}/S$$

V and S being both in tons, S is taken at 10 000‡ lb or 5 tons p
Table II, page 703.)

The MAXIMUM VERTICAL SHEAR in any beam or girder is at the
and is equal to it.

For a girder supported at both ends and uniformly loaded
the MAXIMUM VERTICAL SHEAR is

$$V_{\max} = W/2$$

For a girder supported at both ends and loaded at the middl

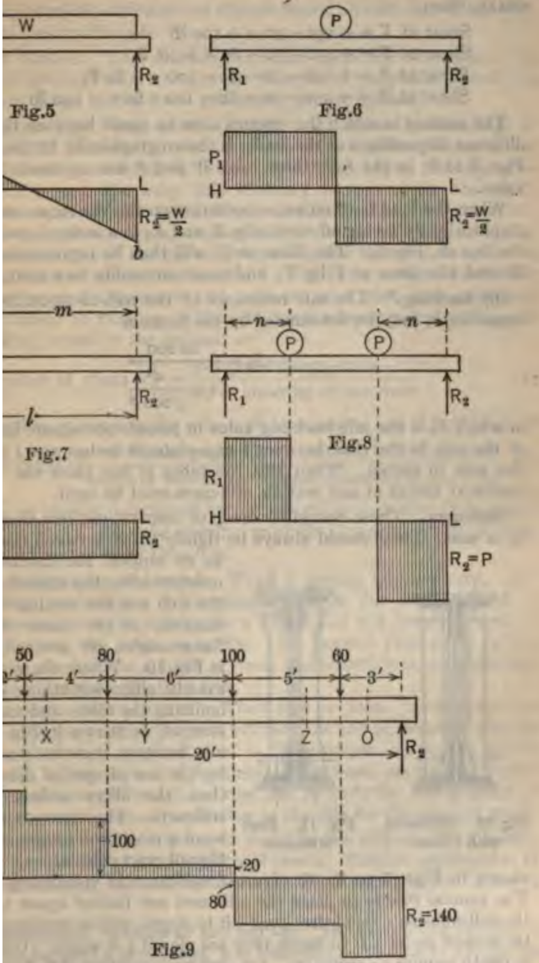
$$V_{\max} = P/2$$

For a girder supported at both ends and loaded as in Fig. 7,

$$V_{\max} = Pm/l = R_1$$

For a girder supported at both ends and loaded with two equ
loads P, P, equally distant from the middle, as in Fig. 8,

en vertical section of a beam or girder between the supports is of all the vertical external forces acting on the beam to the



9. Diagrams for Vertical Shears for Different Loadings

on, forces acting upwards being considered as plus, and those s being considered as minus.

ase of the beam shown in Fig. 9, the REACTION R_1 will be thod explained in Example *2, page 323, and by Formulas (2)*

and (3), page 323, to be 150 lb, and that at R_2 to be 140 lb. The sh
ous sections may be found by applying the foregoing definition o
SHEAR, thus:

Shear at $X = +150 - 50 = +100$ lb
Shear at $Y = +150 - 50 - 80 = +20$ lb
Shear at $Z = +150 - 50 - 80 - 100 = -80$ lb
Shear at $O = +150 - 50 - 80 - 100 - 60 = -140$ lb

The manner in which the VERTICAL SHEAR varies between the sup
different dispositions of the loads, is shown graphically by the hatc
Figs. 5 to 9; in the first three cases W and P are assumed to hav
value.

When the load is UNIFORMLY DISTRIBUTED the VERTICAL SHEAR c
graphically by laying off vertically R_1 and R_2 to a scale of pounds,
the line ab, Fig. 5. The shear at X will then be represented by
X_1 and the shear at Y by Y_1, and they can readily be scaled.

(b) **Buckling.*** The safe resistance of the web to BUCKLING, in
square inch, may be determined by the formula

$$S_b = \frac{10\,000}{1 + \dfrac{d^2}{3\,000\,t^2}}$$

in which S_b is the safe buckling value in pounds per square inch, d
of the web in the clear between flange-plates in inches and t is the
the web in inches. When this resistance is less than the UNIT
VERTICAL SHEAR at any section, stiffeners must be used.

Stiffeners. These should be made of ANGLES, not less than 3½
in in size. They should always be tightly fitted between the flang
as to support the horizontal
order to bring the stiffeners in c
the web and the vertical leg of
FILLERS, of the same thickn
flange-angles, are generally use
in Fig. 10. Where there are sev
exactly alike, something may b
omitting the fillers and BENDING
FENERS, as shown in Fig. 11.
ing, however, can be done pro
by the use of special dies, and
than the fillers unless there
stiffeners. The SPACING OF STI
more a matter of judgment and
than of exact calculation. SHEAR
shown in Figs. 5 to 9, are of great assistance in visualizing shear
The general rule is to place the stiffeners not farther apart than th
the full web-plate on girders over 3 ft in depth, with a maximum spa
On girders under 3 ft in depth they are placed 3 ft apart. Girders
in depth require no stiffeners. On girders supporting distributed loa
generally placed nearer together at the ends than towards the middl

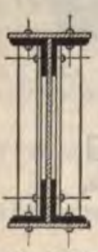


Fig. 10. Stiffeners with Fillers

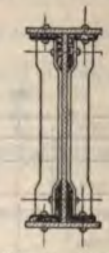


Fig. 11. Bent Stiffeners

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* See Table III, page 705, and also in Chapter XV, the paragraphs a
pages 567 to 569, relating to the web-buckling of beams and girders.
used for web-buckling in Table III, page 705, is the formula that was used i
Steel Company's Manual, and as the values computed by it vary but littl
deduced by the Cambria formula (see page 568), Table III is retained as

rs should always be placed at the ends of girders and directly over the
ıch support, as shown in Fig. 18, and wherever there are concentrated
n plate girders the stiffeners are always placed on each side of the web;
rders on the outside only.

aring of Girders. This depends somewhat upon the character of the
ut a safe general rule is to make the BEARING of the girder beyond the
e support equal to ONE-HALF THE DEPTH OF THE GIRDER.

e Number and Pitch of the Rivets. **(a) Rivets in Web-Legs of**
It will readily be seen that when a plate or a box girder is loaded, the
of the bending moments is to cause the flange-plates and angles to
RIZONTALLY past the web; this tendency is resisted by the rivets
nect the angles with the web. The TOTAL AMOUNT OF THIS TENDENCY
called the HORIZONTAL FLANGE-STRESS, between any section of the
d the nearer end of the girder, is equal to the BENDING MOMENT at
DIVIDED BY THE DEPTH OF THE WEB.* The TOTAL NUMBER OF RIVETS
that section and the nearer end must be such that their combined
to SHEARING OR BEARING, whichever has the lower value, shall equal
ontal flange-stress at the section; or

$$\text{number of rivets} = \frac{\text{horizontal flange-stress}}{\text{bearing or shearing of one rivet}} \tag{5}$$

otal number of rivets in the web-angle from end to end is twice this, or

$$\text{nber of rivets} = \frac{2 \times \text{maximum bending moment† in foot-pounds}}{\text{depth of web in feet} \times \text{least resistance of one rivet}} \tag{6}$$

UMBER OF RIVETS determined by formula (6) is such that they would
than 6 in apart, then the number must be increased, as in no case
ey have a greater PITCH than 6 in.

ets in Flange-Legs of Angles. WITH A SINGLE COVER-PLATE. For
th a single cover-plate, it is customary to put the same number of
he flange-leg as in the web-leg for a distance of 3 ft from the ends of
, STAGGERING the rivets as in Fig. 15. Beyond that point to the
the girder one-half the number of rivets will be sufficient, provided this
ive them a greater pitch than 6 in.

TWO OR MORE COVER-PLATES. When two or more cover-plates are
plate must have sufficient rivets between the end of the plate and the
re its resistance is required, that is, for example, between *a* and *b*,
o transfer to the angle and flange-plates between, an amount EQUAL
FE STRENGTH OF THE PLATE. From this point to the middle point of
, the rivets can be spaced according to the rule for the greatest pitch.

ets in Stiffeners. The spacing of rivets in the stiffeners is generally
d by the rules given for the pitch of rivets. Further explanation of
od of determining the spacing of rivets will be found in the following

e Approximate Weight of the Girder. In determining the size of
girder to support a given load, it is desirable to be able to add to the

---

ormula (1), page 683, and foot-note relating to it. $M = SAd$, and hence
d, SA being the total amount, in pounds, of the tendency to slide, and S being
ntal unit, flange, fiber-stress in pounds per square in, due to flexure. A is the
are inches of the cross-section of the flange and d is the approximate depth of

se the maximum *horizontal flange-stress is equal to the maximum bending*
vided by the girder-depth, or $S_{max}A = M_{max}/d$.

(5) The Determination of the Lengths of the Flange-
methods used to determine these, see the following examples.

4. Explanation of Tables

The Calculations for the Design of Riveted Girders
facilitated by the use of Tables I, II, III and IV at the end of

Table I gives the sectional area that should be deducted f
plates of different thicknesses. In computing this table ⅛ in
diameter of the rivet to allow for the injury to the metal cau
and also to allow for the expansion of the heated rivet.

Table II gives the safe shearing value for web-plates for var
thicknesses, and the deduction to be made for each ¾-in or ⅞-i

Table III gives the safe resistance to buckling per square inc
and also the total safe resistance in pounds for the more comm
plates, with two rivet-holes deducted. It is very seldom that
tion between the stiffeners contains more than two rivet-holes.
the dimensions and properties of angles will be found in Tabl
pages 362 to 367, and the shearing value and bearing values of
in Tables II and III, pages 418 and 419.

Table IV gives the elements of riveted plate girders of vario
which it is possible to select economical sections for almost any
tion of loading.

5. Examples of Plate and Box Girders

Example 1. It is required to support the floor over a room
means of riveted steel plate girders, placed across the room, 1
The room above is to be used for general assembly purposes.
are of wood and there is a plaster ceiling on the under side of the
of the girder is required.

First Step. The Load. The first step is to determine the load

Step. **The Flange-Area.** The next step is to determine the flange-
ore this can be done, however, the width and depth of the girder
ecided. As it is desirable to keep the girder as shallow as possible.
with good engineering, the case will be considered an exceptional
ie depth of the web-plate will be made 36 in, which is about one-
he span and a little less than the usual limit.
irders are braced sidewise by the floor-joists, it will not be necessary
he width of the flange-plates one-twentieth the span of the girder,
y be made 12 in width. The flange-area may be determined by
), page 683, and is

$$A = M_{\max}/dS$$

$$\text{flange-area (sq in)} = \frac{\text{maximum bending moment (ft-tons)}}{\text{depth of web (ft)} \times S \text{ (tons per sq in)}}$$

ximum bending moment for a uniformly distributed load on a simple
$I_{\max} = Wl/8$ (Case V, page 326), or in this particular case, 53.57
ft/8 = 334.8 ft-tons.
ue of S* has varied in the handbooks from 6 to 8 tons, depending
ing conditions and upon the judgment of engineers. A value of S
r 14 000 lb per sq in is the requirement of the new New York Building

ting these values in the formula gives for the net area of either flange,
7 = 16 sq in.
er flange may now be designed. For a girder of this size and loaded
y, it will be advisable to try two 5 by 3½ by 7/16-in angles, with the
horizontal.† The sectional area of these angles (Table XI, page 363)
n which leaves 9 sq in for the area of the flange-plates. Dividing this
he width of the flange, gives ¾ in for the total thickness of the plates,
be made up of two ⅜-in thick plates. Of course, any other combina-
ates and angles having an area of cross-section of 16 sq in will fulfill
ions of the problem, the selection in all cases depending upon the
and experience of the designer. Note, also, that no part of the web
ncluded in the flange-area although it would be safe to include one-
. This also is a matter of individual opinion.
ower flange is in tension, the rivet-holes should be deducted in order
the net area. Assuming that ¾-in diameter rivets are used, it will
hat the greatest loss of section is by two rivet-holes opposite each
ecting the angles with the plates of the bottom flange. From Table I,
the area of two ¾-in rivets in a ¾-in plate is 1.31 sq in, and in a
e, the same thickness as that of the angles, it is 0.76 sq in. The sum
hicknesses is 2.07 sq in, which must be added to the net area of the
ge-plates, 16 sq in, making 18.07 sq in for the gross area of the lower

---

Chapter XV, paragraphs and foot-notes, page 603, relating to fiber-stresses
for riveted beam girders, etc.

flange-angles of plate girders the 5 by 3½-in size is most commonly used,
nge-plate is 12 in wide, and 6 by 4-in angles when the flange-plate is over 12 in
box girders 5 by 4, 5 by 3½, 4 by 3½ and 3½ by 3½-in are common sizes;
ry heavily loaded girders, requiring two rows of rivets in the web-leg, 6 by
are often used. For most riveted girders, in which only one row of rivets is
short leg is riveted to the web, so as to bring most of the material as far from
axis of the girder as possible. *The minimum thickness of flange-angles
in, and the maximum thickness for ordinary loads is ¾ in.*

flange-plates. This additional area may be obtained by increasing the
ness of the plates to ½ in.

The flanges will then be made up as follows:

Upper flange: Two angles 5 by 3½ by 7/16-in = 7.06 sq in. gros
Two plates 12 by ⅜-in = 9.00 sq in. gros
Total 16.06 sq in. gros

Lower flange: Two angles, 5 by 3½ by 7/16-in = 7.06 sq in. gros
Two plates, 12 by ½-in = 12.00 sq in. gros
Total = 19.06 sq in. gros

Third Step. The Length of the Flange-Plates. To determine th
necessary to plot the bending-moment diagram shown in Fig. 12. The be

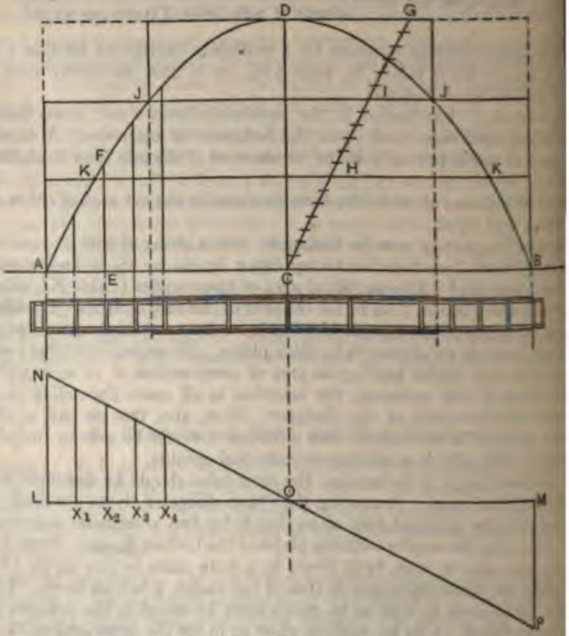


Fig. 12. Diagrams for Bending Moments and Vertical Shears. Example

moment diagram for a girder under a uniformly distributed load is bound
a parabola having a height over the middle of the girder equal to the ma
bending moment. From the middle point *C*, of a horizontal line *AB*,
convenient scale, lay off a vertical line *CD*, equal to the maximum be
moment, 334.8 ft-tons. Construct the parabola *ADB* (see page 79); th
bending moment at any other point, as *E*, is equal to the ordinate *EF*
that point, measured to the same scale.

In practice, however, the plates are usually extended beyond the
l *K* on each side as an additional factor of safety, a distance suffi-
e enough rivets to transmit at least one-third the resistance of the
also customary to make the first or lower flange-plate the full
e girder as it greatly stiffens the angles and adds but a small
the cost. Theoretically the length of the flange-plates of the top
l be less than the length of the plates of the lower flange, because
ea of the top flange is less than that of the lower flange; but they
made the same length.

ep. The Web. Webs are proportioned to resist the shear. The
earing-stress in a girder uniformly loaded is equal to either reaction,
s case is one-half the total load, or 53 500 lb. As the girder is 3 ft
nall shear would require a very thin section, thinner than the min-
ess for webs, which is ⅜ in. From Table II, page 703, it is seen
aring resistance of a ⅜ by 36-in web-plate is 135 000 lb, which is
cess of the actual shear.

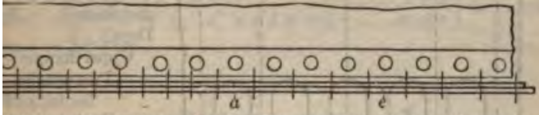
. **The Stiffeners.** As before explained, stiffeners will be required
e vertical shear exceeds the safe resistance of the web to buckling.
shear is 53 500 lb and the resistance to buckling may be found from
age 705. This, for a ⅜ by 36-in web with two ¾-in rivets is found
lb; hence stiffeners will have to be used. As stated under Buckling
e 686, the spacing of stiffeners is more a matter of judgment and
han of exact calculation, and for this a shear-diagram, also shown
is of great assistance. It may be constructed as follows: On a
ne *LM*, lay off to any convenient scale vertical lines *LN* and *MP*,
nting 53 500 lb. Connect the points *N* and *P*; then the vertical
point is equal to the ordinate at that point, measured to the same
, at X_1, 3 ft from the left end, the shear is 47 500 lb, at X_2, 6 ft
40 500 lb, at X_3, 9 ft from *L*, it is 34 000 lb and at X_4, 12 ft from
o lb. As the vertical shear at X_3 is greater than the safe resist-
kling and at X_4 less, it might be safe to stop the stiffeners at X_4;
or-joists are framed flush, or nearly so, with the top of the girder,
on angles riveted to its web, it will be advisable to put about 3

25 tons = 50 000 lb. The number of rivets required between th left reaction is, from Formula (5), equal to 50 000/5 630 = 10 r to be spaced in a distance of 36 in, making the spacing 3.6 in. bending moment scales 141.24 ft-tons, the flange-stress is 141.2 or 94 160 lb, and the number of rivets required between X_2 and = 17; but 10 of these are required between X_1 and A, leavin between X_1 and X_2 in a distance of 36 in making the spacing 5. bending moment scales 197.4 ft-tons, and the flange-stress is 197. or 130 600 lb. The number of rivets required is 130 600/5 630 these are required between X_2 and A, leaving 7 to be placed X_3, making the spacing the same as in the second panel. At moment scales 243.96 ft-tons, and the flange-stress is 243.96/ 162 640 lb. The number of rivets required is 162 640/5 630 these are required between X_3 and A, leaving 6 to be placed bet in a distance of 36 in, making the spacing 6 in. As this is the m allowed, it will be used from X_4 to the corresponding point right-hand half of the girder. The same number of rivets wil flange-legs of the angles as in the web-legs, but they will be they will come between those in the web.

The outer flange-plate scales 28 ft 6 in in length in the bendi gram, but this length, as before stated, should be increased suf enough rivets to transmit at least one-third of the resistance. plate is ½ in × 12 in = 6 sq in, minus the area of two ¾-in riv in (Table I, page 702), leaving a net area of 5.13 sq in. T the plate is therefore equal to 5.13 sq in × 14 000 lb per sq in = third of this, or 23 940 lb, must be transferred by rivets pla points JJ. As the rivets in the flange are in single shear, th of one rivet in single shear, 4 420 lb, will govern. The numb quired, then, is 23 940/4 420 = 6, or 3 in each angle. The spac in this panel is 6 in. The plates will therefore be extended 18 of JJ.

* The shearing value of rivets is taken at from 7 000 to 12 000 lb per ing value at from 12 000 to 24 000 lb per sq in. The usual values are

ιs the total length of the girder is but 53 ft, it will not be necessary webs or the flanges, because the extreme length of a ⅜ by 36-in 't and of a 12 by ½-in plate, 90 ft.* It is never necessary to splice ey are rolled in lengths up to 90 ft. In very long, deep girders, s sometimes necessary to splice the web, and the joint is sometimes niddle, as theoretically there is no vertical shearing-stress in the web



Splicing of Inner Plate of Bottom Flange of Plate Girder. Example 1

t when the load is uniformly distributed. Generally, however, pliced in two places, equidistant from the middle of the girder. calculated for vertical shear only, the rule being to divide the shear by the safe shearing value or bearing value of one rivet. This nber of rivets required on each side of the splice-plate, unless the ch is exceeded, when more are added.

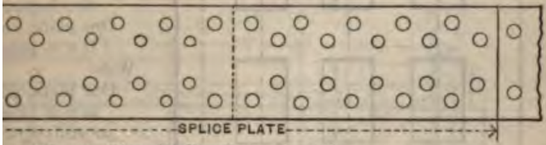
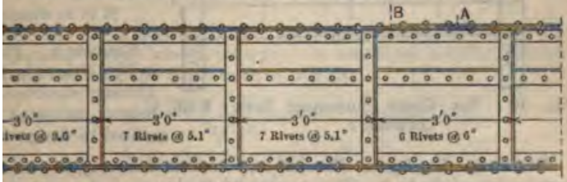


Fig. 14. Plan of Splice-plate. Example 1

a splice is required in a flange-plate, it should be, if possible, at a yond the end of the plate above it. The joint must be made by e spliced plate, a plate of the same thickness and of sufficient length number of rivets on each side of the joint equal to the strength of t is spliced. When the flange is made up of two plates of the same simplest method of splicing the inner plate is as shown in Fig. 13.



ig. 15. Elevation of Part of Plate Girder. Example 1

the theoretical position of the end of the outer plate, as determined ng-moment diagram, and *a* the point to which the plate must be receive rivets of a resistance equal to one-third the strength of hen let the joint in the inner plate be just over *a* and extend the *b*, or such a distance that it can receive a number of rivets equal to the strength of one plate.

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extreme lengths are published in the various handbooks. The above example, are taken from the table on page 103 of Carnegie's Pocket Com-dition.

Fig. 15 shows one end of the girder, drawn according to the fore
lations.

The Bill of Quantities for the Girder. The following is a bill of
for the construction
girder.

Load: 100 000 lb,
distributed. Span
Depth 3 ft

Upper flange:
- Two angles, 5 b
  $\frac{7}{16}$ in, 53 ft lo
- One plate 12 by
  ft 0 in long
- One plate, 12 by
  ft 6 in long

Lower flange:
- Two angles, 5
  $\frac{7}{16}$ in, 53 ft lo
- One plate, 12 by
  ft 0 in long
- One plate, 12 by
  ft 6 in long

Web:
- One plate, 36 by
  ft 0 in long
- 30 stiffeners,
  $\frac{3}{8}$-in angles,
  long
- 30 filler-plates,
  29 in long
- 92 ft 8 in of
  $\frac{1}{2}$-in angles
  porting floor-

Rivets:
- $\frac{3}{4}$ in in diamete

Example 2. The
in Fig. 16 is to be sup
a riveted-steel box gi
height indicated.
quired to design the

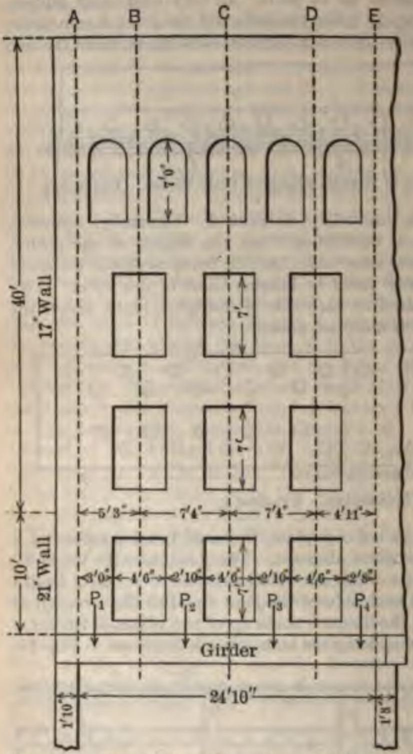


Fig. 16. Box Girder Supporting Brick Wall. Example 2

First Step. The Load. The first step towards designing the girder
termination of the load. The space under the lower windows is too sm
tribute the weight from the piers uniformly over the girder, so that th
assumption is that the weight of the wall between the lines *A* and *B*
trated at P_1, the weight of wall between lines *B* and *C* at P_2 and so
floor-joists run across the building, so that only the weight of the
supported by the girder. Allowing 200 lb per square foot of face for
wall, and 165 lb for the 17-in wall, both walls being plastered on the
Load at P_1

$$= \left\{ \begin{array}{l} [5'\,3'' \times 10' - 7' \times 2'\,3''] \times 200 = \dots\dots\dots\dots\dots\dots 7\,350 \\ [5'\,3'' \times 40' - (2'\,3'' \times 14' + 3'\,2'' \times 7')] \times 165 = \dots\dots 25\,795 \end{array} \right\} =$$

P_2

$$\left.\begin{array}{l} '' \times 10' - 4'6'' \times 7'0''] \times 200 = \ldots\ldots\ldots\ldots\ldots\ 8\,366 \\ '' \times 40' - (4'6'' \times 14' + 4'9'' \times 7')] \times 165 = \ldots\ldots 32\,354 \end{array}\right\} = 40\,720 \text{ lb}$$

P_3 = that at P_2 = ... 40 720 lb

P_4

$$\left.\begin{array}{l} 1'' \times 10' - 2'3'' \times 7] \times 200 = \ldots\ldots\ldots\ldots\ldots\ 6\,683 \\ 1'' \times 40' - (2'3'' \times 14' + 3'2'' \times 7')] \times 165 = \ldots\ldots 23\,595 \end{array}\right\} = 30\,278 \text{ lb}$$

otal load on girder = ... 144 863 lb

ons

uation (7)

imate weight of girder $= \dfrac{W \times l}{700} = \dfrac{72.4 \times 24\frac{5}{6}}{700} = 2.5$ tons, or 5 000 lb

e-third of this, or say 1 600 lb, should be added to P_2 and P_3, and 900 lb P_4. This will give, approximately, the following loads, applied as in

$P_1 = 34\,000$ lb $\quad P_2 = 42\,300$ lb

$P_3 = 42\,300$ lb $\quad P_4 = 31\,200$ lb

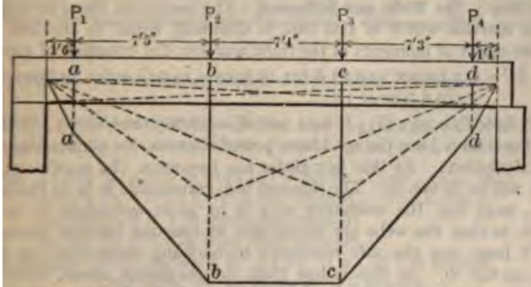


Fig. 17. Diagram for Bending Moments. Example 2

Step. The Determination of the Maximum Bending Moment. By the formula under Case VI, page 327, the maximum bending moment ounds for the loads are found to be as follows:

$$\text{For } P_1,\ M_{\max} = \frac{34\,000 \times 1'6'' \times 23'4''}{24'10''} = 47\,980 \text{ ft-lb}$$

$$\text{For } P_2,\ M_{\max} = \frac{42\,300 \times 8'11'' \times 15'11''}{24'10''} = 242\,000 \text{ ft-lb}$$

$$\text{For } P_3,\ M_{\max} = \frac{42\,300 \times 16'3'' \times 8'7''}{24'10''} = 237\,900 \text{ ft-lb}$$

$$\text{For } P_4,\ M_{\max} = \frac{31\,200 \times 1'4'' \times 23'6''}{24'10''} = 39\,420 \text{ ft-lb}$$

these moments to a scale, as explained for Fig. 15, page 329, the moment diagram shown in Fig. 17 * is obtained. The maximum bend-

---

ending moments in this diagram are drawn to a scale of about *400 000 ft-lb* h.

As the thickness of the wall to be supported is 21 in, the flan
at least 20 in wide and not less than ⅜ in thick. The sectio
by 20-in plate is 7½ sq in, leaving 4½ sq in to be made u
The sectional area of two 5 by 3½ by 7/16-in angles is 7.06 s
page 363), which leaves a small excess for the lower flange.
in diameter, the loss in area due to two rivet-holes in a ⅜-in
page 702) 0.65 sq in and in a 7/16-in plate, the thickness of each
making 1.41 sq in in all, for which the excess in the angles is
cient. The width of the flange being more than one-twentieth
lateral support unnecessary.

Fourth Step. The Webs and Stiffeners. The maximum shea
maximum reaction which in this case is obviously equal to t
Taking the center of moments at the right reaction, the equatio

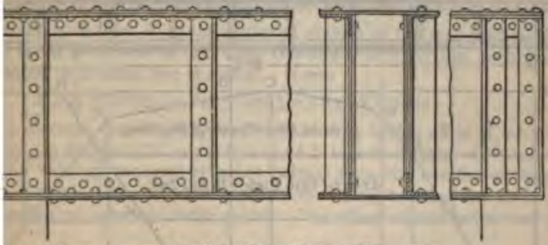
$$R_1 \times 24.83' = (17 \text{ tons} \times 23.33') + (21.15 \text{ tons} \times 15.91') + (21.1$$
$$+ (15.6 \text{ tons} \times 1.33')$$

whence $24.83 \times R_1 = 935.3215$ ft-tons and $R_1 = 37.669$ tons, or 7
that the loads have been changed from pounds to tons, for conv
ing the calculations. As this box girder has two webs, the ma
each web will be 37 669 lb. The thinnest web permissible is ⅜
Table II, page 703, the resistance of a ⅜ by 30-in web-plat
112 500 lb, so that the webs are amply safe in resisting vertic
Table III, page 705, the safe resistance to buckling, deductin
rivets, is 33 830 lb. As this is less than the maximum shea
be used, placed 2 ft 4 in from each support, with five betwee
the spacing about 3 ft 4 in on centers. Two others will be p
support. 4 by 4 by ⅜-in angles will be sufficient for the stiff

Note. If the loads were really concentrated at the points
from columns or girders, it would be necessary to place stiffen
of these points and two in each of the intermediate spaces, b
loads are partly distributed it will be better to space them as fi

Fifth Step. The Number and Pitch of the Rivets. The ri
legs and flange-legs of the angles are in single shear. From
410 the shearing value of a ¾-in rivet in single shear at

for the flange-stress, or 22 600 lb for each web. The number of ore (Formula (5), page 687) is 22 600/4 420 = 6. The distance the left reaction is 18 in, which makes the spacing 3 in. The at P_2 is 209.88 ft-tons/2.5 ft = 83.95 tons, or 167 900 lb, and one- s 83 950 lb. The number of rivets therefore is 83 950/4 420 = 19. se are required between P_1 and the left reaction, leaving 13 to go nd P_2, a distance of 89 in, making the pitch about 6.9 in. As the maximum allowable pitch, the rivets will be spaced 6 in on een P_1 and P_2, and between P_2 and P_3. The spacing on the right- the girder will be made the same as that on the left. Some details are shown in Fig. 18.



. 18. Elevations and Section of Box Girder. Example 2

and Bill of Quantities for the Girder. The loads, dimensions, size, ces, etc., for the girder are given in the following summary:

4 000 lb, 1 ft 6 in from left support. Span: 24 ft 10 in
2 300 lb, 8 ft 11 in from left support. Depth: 30 in
2 300 lb, 8 ft 7 in from right support
1 200 lb, 1 ft 4 in from right support
ges: Four angles, 5 by 3½ by $\frac{7}{16}$ in, 27 ft 6 in long
One plate, 20 by $\frac{3}{8}$ in, 27 ft 6 in long
s: $\frac{3}{8}$ by 30 in, 27 ft 6 in long
two stiffeners: 4 by 4 by $\frac{3}{8}$ in, 29$\frac{1}{8}$ in long
two filler-plates: 4 by $\frac{7}{16}$ in, 23 in long
$\frac{3}{4}$ in in diameter

What are the dimensions of a box girder, 40 ft in span, required to llowing loads? 90 tons from a column, 8 ft from the left support; a column, 12 ft from the right support; and a masonry pier, 10 ft inning 10 ft from the left support and weighing 4 tons per running 19.)

The Determination of the Reactions, Shears and Bending Mo- nd either reaction, the center of moments is taken at the other equation of moments for the left reaction is, therefore, taking the nents at the right reaction,

$$= (90 \text{ tons} \times 32 \text{ ft}) + (40 \text{ tons} \times 25 \text{ ft}^*) + (75 \text{ tons} \times 12 \text{ ft})$$

$$40 R_1 = 4\,780 \text{ ft-tons and } R_1 = 119.5 \text{ tons}$$

---

ing the moments of forces, distributed loads are treated as if they were their centers of gravity.

In like manner, the equation of moments for the right reaction is

$$40\,R_2 = (75 \text{ tons} \times 28 \text{ ft}) + (40 \text{ tons} \times 15 \text{ ft}) + (90 \text{ tons} \times 8 \text{ ft})$$

from which

$$40\,R_2 = 3\,420 \text{ ft-tons, and } R_2 = 85.5 \text{ tons}$$

The greatest vertical shear V_1 is equal to the greater reaction, 119.5 tons. The shear-diagram (Fig. 19) may be constructed by layi

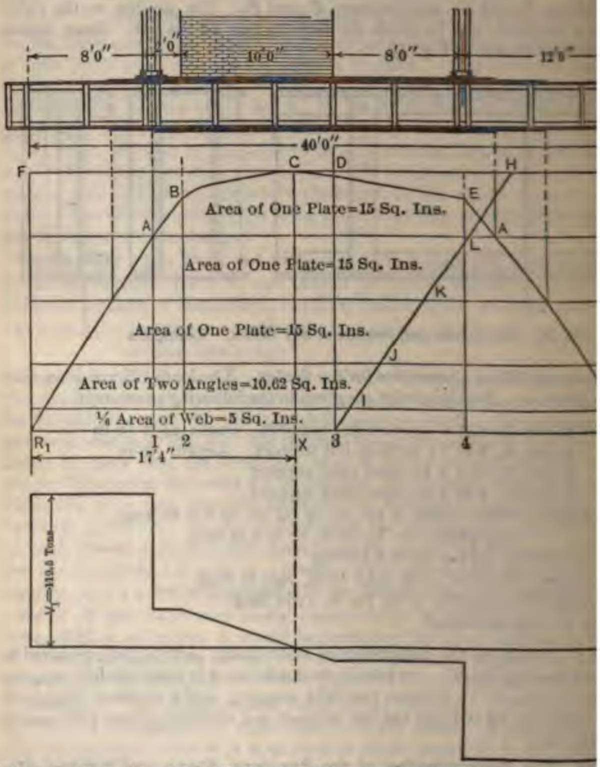

8'0"
1'0"
10'0"
8'0"
12'0"
40'0"
F
A
B
C
D
E
H
A
L
K
J
I
Area of One Plate=15 Sq. Ins.
Area of One Plate=15 Sq. Ins.
Area of One Plate=15 Sq. Ins.
Area of Two Angles=10.62 Sq. Ins.
¼ Area of Web=5 Sq. Ins.
R₁
1
2
X
3
4
17'4"
V₁=119.5 Tons

Fig. 19. Elevation of Box Girder and Diagrams for Bending Moments a
Shears. Example 3

any convenient scale an ordinate equal in length to 119.5 tons. Im at the right of point 1, under the left column, the shear is equal to 11 29.5 tons. It is the same at point 2, the left end of the wall. At po right end of the wall, the shear is $119.5 - 90 - 40 = -10.5$ tons, showin shear passes through zero somewhere between 2 and 3, which is the maximum bending moment. This point, X, is by scaling the shear 17.4 ft from R_1. It is over the point of intersection of the slanti considering the forces to the left of X, is

.5 tons × 17.4 ft) – (90 tons × 9.4 ft) – (4 tons × 7.4 ft × 7.4 ft/2)

e distance from X to the center of gravity of the wall-load to the
d is the lever-arm of that load, considered as a vertical downward
rated in a single line of action. Hence

$$M_{\max} = 2\,079.3 - 846 - 109.5 = 1\,123.8 \text{ ft-tons}$$

g-moment diagram may be constructed by laying off at X, at any
ale, an ordinate XC equal to 1 123.8 ft-tons in length. It is neces-
the bending moment at other points, since the bending-moment
not be plotted, as in the previous examples, because the uniform
istributed over the entire girder. The other critical points are 1,

5 tons × 8 ft) = 956 ft-tons
5 tons × 10 ft) – (90 tons × 2 ft) = 1 015 ft-tons
5 tons × 20 ft) – (90 tons × 12 ft) – (40 tons × 5 ft) = 1 110 ft-tons
5 tons × 28 ft) – (90 tons × 20 ft) – (40 tons × 13 ft) = 1 026 ft-tons

off ordinates at these points equal by scale to the respective bending
rawing straight lines from R to A, the extremity of the ordinate
nd from A to B; drawing curved lines from B through the points
d connecting D and E and E and R_2 by straight lines; the bending-
ram $R_1ABCDER_2$ may be constructed.

ep. The Webs. As stated on page 683, it is considered safe by
ers to include one-sixth of the web-area in the flange-area, and
lone in this example. The web, therefore, must be designed first.
othing to limit the depth of the girder, it will be made 3 ft deep,
elfth the span. The greatest vertical shear is equal to the greater
on, 119.5 tons. Since the girder carries a brick wall, it must be of
, and hence the vertical shear on each web is 59.75 tons. A ½ by
ll be tried first. Its area is 18 sq in, from which must be deducted
ea due to the rivet-holes for the rivets through the stiffeners. The
e placed the maximum distance on centers, making six in each

To proportion the lower flange, allowance must be made for the loss in area due to two rivet-holes.

From Table I, page 702, the area of two $\frac{7}{8}$-in rivet-holes in a $\frac{9}{16}$-in plate (thickness of angles) = ...

Area† of two rivet-holes in three $\frac{5}{8}$-in flange-plates = ...

Total rivet-area = ...

Hence the gross section-area of the lower flange must be $53.5 + 4.87 = 58$ sq in.

This may be made up of

One-sixth of net section-area of two webs = ...

Two 5 by 5 by $\frac{9}{16}$-in angles, with section-area = ...

Three $\frac{5}{8}$ by 24-in plates, with section-area of 15 sq in each = ...

Total section-area of lower flange‡ = ...

The length of the flange-plates is determined from the diagram. Draw a horizontal line through *C* (Fig. 19) and at lay off to any convenient scale and angle, a line $3H = 60.62$ with its upper extremity on the horizontal line *FG* drawn thr this line into five parts: $3I$, containing 5 units for the web-area for the angles; and *JK*, *KL* and *LH* of 15 units each, for the thr horizontal lines through the points *I*, *J*, *K* and *L* as shown. intercepts of these horizontals in the bending-moment diagra theoretical lengths of the flange-plates. For practical consider plate is always carried the full length of the girder and the oth tended beyond the intersection-points on either side, a dista take enough rivets to transmit at least one-third of the resista The resistance *AS*, of the outer plate is 15 sq in × 14 000 lb per s One-third of this, or 70 000 lb, must be resisted by rivets pl points *AA*. From Table III, page 419, at 10 000 lb per sq value of a $\frac{3}{4}$-in rivet in single shear plate is 6 010 lb and in a $\frac{7}{8}$-in value at 18 000 lb per sq in is 9 820 lb. Hence the number of

. **The Stiffeners.** From Table III, page 705, the safe buckling
by 36-in plate with two ⅞-in rivets is 62 320 lb, and as this is
an the shearing value, stiffeners must be used. The stiffeners
centrated loads may be considered as short struts in direct com-
suming that 4 by 4 by ½-in angles are used for the stiffeners, the
n Table XV, page 502, is over 20 000 lb. The greatest concen-
90 000 lb, and hence four stiffeners will be placed under each
r more will be placed at each bearing, as shown in Fig. 19, four
etween the columns, about 4 ft on centers; and two on each side,
olumns and the bearings, making 15 on each side, or 30 in all.

The Number and Pitch of the Rivets. In a box girder, the rivets
hear. The shearing value of a ⅞-in rivet at 10 000 lb per sq in
e III, page 419, 6 010 lb, and its bearing value at 18 000 lb per
in plate, the thinnest outside plate, is 6 880 lb; hence the shearing
ern.

r of rivets depends upon the horizontal flange-stress, which is
maximum bending moment divided by the depth of the girder
page 683). *M* at 1 = 956 ft-tons, and the horizontal flange-stress
9 tons, or 638 000 lb. From Formula (5), page 687, the number
ired = 638 000/6 010 = 106, or 53 on each side. These are to be
istance of 8 ft, or 96 in, which makes the pitch about 1.8 in. As
an the minimum pitch, 2⅝ in, or three diameters, the rivets will
aggered. Hence the justification for selecting large angles with
r this paricular girder. At *X* the horizontal flange-stress =
4.6 tons, or 749 200 lb, and the number of rivets is 749 200/6 010
on each side; 53 of these, however, are required between R_1 and 1,
e placed between 1 and *X*, a distance of about 9 ft. As the re-
will exceed the maximum pitch, they will be placed 6 in on centers
d *X*. At 4 the horizontal flange-stress = 1 026/3 = 342 tons, or
The number of rivets is 684 000/6 010 = 112, or 56 on each side,
in a distance of 12 ft, or 144 in, making the spacing 2.5 in. Be-
the maximum pitch will be determined as before.

The Weight of the Girder. So far, no account has been taken
of the girder. The practice is to neglect this weight when the max-
moment due to it alone is less than 10% of the maximum bend-
lue to the loads. From Formula (7), page 688, the weight of the
40/700 = 12 tons. From Case V, page 326, the maximum bend-
lue to it = 12 × 40/8 = 60 ft-tons. As this is much less than 10%
tons, the maximum bending moment due to the loads, it may be
Had it been otherwise, the weight would have to be considered as
uniformly distributed load over the entire girder and a new bend-
diagram drawn.

a on Riveted Girders. By applying the principles illustrated in
examples it is possible to compute the necessary dimensions and
eted girders under any conditions of loading. If further examples
he reader is referred to "Compound Riveted Girders," by William
in which different examples of loading are fully worked and ex-
also to other recent treatises on this subject.

wings and Stress-Diagrams of one of the earlier heavy plate
n building-construction are published in the Engineering Record
395. This girder is one of six plate girders used in the construc-
nt Temple, Boston, Mass., Blackall & Newton, architects. The
long between centers of columns, 6 ft 1 in deep, with flanges 28 in

wide, and is calculated to support distributed and concentrated loa
gating 497.5 tons. The single web-plate is 64¾ in deep, and ⅝ in thi
ends; the flanges are 4½ in thick at the middle of the girder; and the
angles are 6 by 8 by 1 in. Since that time there have been erected for
the large buildings a number of riveted girders of very great size and
and details of their construction may be found in the engineering and
tural periodicals.

6. Tables Used in the Design of Plate and Box Girder

Tables I, II, III and IV contain data usually required for the desig
and box girders to satisfy all but the most unusual conditions.

Table I.*† Sectional Area in Square Inches to be Deducted from Pl Angles for Rivet-Holes

Taken ⅛ inch in excess of diameter of rivet‡

Thickness of plate, in	Number of rivets, 1 in diameter				Number of rivets, ⅞ diameter		
	1	2	3	4	1	2	3
1	1.12	2.25	3.37	4.50	1.00	2.00	3.00
15/16	1.05	2.10	3.16	4.21	0.94	1.87	2.81
⅞	0.98	1.97	2.95	3.93	0.87	1.75	2.62
13/16	0.91	1.83	2.74	3.65	0.81	1.62	2.44
¾	0.84	1.69	2.53	3.37	0.75	1.50	2.25
11/16	0.77	1.55	2.32	3.09	0.69	1.37	2.06
⅝	0.70	1.41	2.11	2.81	0.62	1.25	1.87
9/16	0.63	1.26	1.90	2.53	0.56	1.12	1.69
½	0.56	1.11	1.69	2.25	0.50	1.00	1.50
7/16	0.49	0.98	1.47	1.97	0.44	0.87	1.31
⅜	0.42	0.84	1.26	1.69	0.37	0.75	1.12

Thickness of plate, in	Number of rivets, ¾ in diameter				Number of rivets, ⅝ diameter		
	1	2	3	4	1	2	3
1	0.87	1.75	2.62	3.50	0.75	1.50	2.25
15/16	0.82	1.64	2.46	3.28	0.70	1.40	2.11
⅞	0.77	1.53	2.30	3.06	0.65	1.31	1.96
13/16	0.71	1.42	2.13	2.84	0.61	1.22	1.83
¾	0.66	1.31	1.96	2.62	0.56	1.12	1.69
11/16	0.60	1.20	1.80	2.40	0.51	1.03	1.54
⅝	0.55	1.09	1.64	2.19	0.47	0.94	1.41
9/16	0.49	0.98	1.48	1.96	0.42	0.84	1.26
½	0.43	0.87	1.31	1.75	0.37	0.75	1.12
7/16	0.38	0.76	1.15	1.53	0.33	0.66	0.98
⅜	0.32	0.65	0.98	1.31	0.28	0.56	0.84
5/16	0.27	0.55	0.82	1.09	0.23	0.47	0.70
¼	0.22	0.44	0.66	0.87	0.18	0.37	0.56

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* For explanation of tables, see Subdivision 4, page 688.

† This table is taken from "Compound Riveted Girders," by W. H. Birk

‡ See paragraph, Punching Rivet-Holes, page 414, and Table XI, page

ble II.* Safe Shearing Value of Web-Plates in Pounds

Mild steel. Gross area. Safe unit stress, 10 000 lb per sq in

Thickness in inches						
3⁄8	7⁄16	1⁄2	9⁄16	5⁄8	3⁄4	7⁄8
5 000	122 500	140 000	157 500	175 000	210 000	245 000
2 500	131 300	150 000	168 800	187 500	225 000	262 500
0 000	140 000	160 000	180 000	200 000	240 000	280 000
5 000	157 500	180 000	202 500	225 000	270 000	315 000
0 000	175 000	200 000	225 000	250 000	300 000	350 000
7 500	183 800	210 000	236 300	262 500	315 000	367 500
2 500	201 300	230 000	258 800	287 500	345 000	402 500
0 000	210 000	240 000	270 000	300 000	360 000	420 000
Deductions in pounds for one 3⁄4-in rivet†						
3 200	3 800	4 300	4 900	5 500	6 600	7 700
Deductions in pounds for one 7⁄8-in rivet†						
3 700	4 400	5 000	5 600	6 200	7 500	8 700

ation of tables, see Subdivision 4, page 688.

of the hole is taken 1⁄8 in in excess of the diameter of the rivet to allow for etal sustained by punching.

. What is the safe shearing value of a 36 by 3⁄8-in web-plate with ivets in the stiffeners?

The gross shearing value = 135 000 lb
tion for seven rivets = 7 × 3 200 = 22 400 lb
he safe shearing value = 112 600 lb

able for any other unit stress, divide the shearing value by 10 000 by the given unit stress. For example, what is the safe shearing-o by 5⁄8-in web-plate at 12 000 lb per sq in? (250 000/10) × 12 =

f Riveted Steel Plate Girders.‡ It is not practicable to give AFE LOADS for riveted steel plate girders because of the great variety ons of plates and angles that can be selected for any given condition Moreover, any variation in the loading would make the tables use-ce of the safe loads, therefore, the PROPERTIES OR ELEMENTS OF EL PLATE GIRDERS are given in Table IV, pages 706 to 716, which etermining the size of the girder and the approximate thickness of d angles for any special case. To determine the dimensions and of a girder suitable to carry any specified loading, determine the D-REACTION in pounds and the MAXIMUM BENDING MOMENT in inch-lect from Table IV the different parts for a girder of the required ICKNESS OF WEB as determined by the maximum end-reaction and CTION-MODULUS as determined by dividing the maximum bending

s of riveted single-beam girders and double-beam *girders, see Tables* XIV s 605 to 611.

modulus $I/c = M/S = 8\,033\,000/14\,000 = 574$. The depth o limited to 36 in. Looking up the properties of 36-in girders in 709, it is seen that a ⅜-in web is more than sufficient to reaction. The nearest section-modulus to 574 is 567.2, that posed of a 36 by 7/16-in web, 5 by 3½ by ½-in angles, and 12 plates. In working out the problem in detail it was found required 5 by 3½ by 7/16-in angles and two 12 by ½-in flang pensate for the loss of area due to the punching of the rivet-ho

Table III.* Safe Buckling Values of Web-Plates

SAFE UNIT BUCKLING VALUE IN POUNDS PER SQUARE INCH

$$\text{Calculated by formula}\dagger\ S_b = \frac{10\,000}{1 + \dfrac{d^2}{3\,000\,t^2}}$$

afe buckling resistance in pounds per square inch; d = depth of web in the clear flange-plates in inches; t = thickness of web in inches

Thickness in inches						
⅜	7/16	½	9/16	⅝	¾	⅞
3 498	4 228	4 890	5 476	5 932	....	....
3 192	3 896	4 546	5 133	5 656	6 522	....
2 889	3 624	4 228	4 787	5 339	6 226	6 920
2 456	3 069	3 666	4 229	4 748	5 656	6 392
2 087	2 696	3 191	3 724	4 228	5 133	5 882
1 930	2 455	2 983	3 498	3 992	4 889	5 649
1 548	1 994	2 543	2 918	3 371	4 228	4 992

OTAL SAFE RESISTANCE IN POUNDS FOR PLATES WITH TWO ¾-IN RIVETS

Thickness in inches						
⅜	7/16	½	9/16	⅝	¾	⅞
34 450	48 580	64 200	80 880	97 340	....	....
33 830	48 150	64 230	81 560	99 880	138 200	....
31 560	46 000	62 800	81 500	101 750	145 300	191 570
29 140	43 230	60 040	79 190	100 440	147 600	198 960
26 860	40 360	58 820	75 920	97 450	146 670	202 000

OTAL SAFE RESISTANCE IN POUNDS FOR PLATES WITH TWO ⅞-IN RIVETS

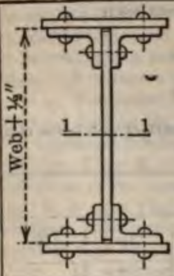
Thickness in inches						
⅜	7/16	½	9/16	⅝	¾	⅞
34 100	48 110	63 570	80 100	96 390	....	....
33 510	47 720	63 640	80 840	98 980	136 960	....
31 310	45 660	62 320	80 900	100 690	144 230	190 170
28 950	42 960	59 660	78 700	99 800	146 690	197 710
26 700	40 140	58 490	75 520	96 910	145 860	200 930

explanation of tables, see Subdivision 4, page 688.

in Chapter XV the paragraphs and foot-notes, pages 568 and 569, relating to buckling of I-beams. The formula for the above table is the formula that was he Passaic Steel Company's Manual, and as the values computed by it vary but n those deduced by the Cambria formula, Table III is retained as it is.

so, page 686, paragraph relating to Safe Resistance of Web to Buckling.

Table IV.* Elements of Riveted Plate Girders



To determine the details of construction of a gi
able to carry any specified loading, determine th
mum end-reactions in pounds and the maximum b
moment in inch-pounds

Select from the table a girder having the desir
a thickness of web as determined by the maxim
reaction and a suitable section-modulus, determ
dividing the maximum bending moment by the p
ble unit bending fiber-stress in pounds per squar

For limiting conditions, see the preceding page
first two subdivisions of this chapter

Weights given do not include stiffeners, rivet-h
other details

Section-modulus, axis, 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
242.0	24×⅜	5×3½×⅝		97.8	
270.9		5×3½×⅜	12×⅜	72.2	30.6
306.1		5×3½×⅜	12×½	72.2	40.8
343.6		5×3½×½	12×½	85.0	40.8
378.5		5×3½×½	12×⅝	85.0	51.0
414.1		5×3½×⅝	12×⅝	97.8	51.0
151.5	26×5⁄16	4×3 ×⅜		61.6	
176.8		5×3½×⅜		69.2	
186.6		4×3 ×½		72.0	
201.2		6×4 ×⅜		76.8	
219.6		5×3½×½		82.0	
252.0		6×4 ×½		92.4	
260.7		5×3½×⅝		94.8	
341.5	26×⅜	6×4 ×⅜	14×⅜	82.4	35.7
354.4		6×4 ×¾		127.6	
377.4		5×3½×½	12×½	87.6	40.8
386.1		6×4 ×⅜	14×½	82.4	47.6
415.2		5×3½×½	12×⅝	87.6	51.0
435.1		6×4 ×½	14×½	98.0	47.6
454.5		5×3½×⅝	12×⅝	100.4	51.0
479.3		6×4 ×½	14×⅝	98.0	59.5
526.1		6×4 ×⅝	14×⅝	113.2	59.5
569.9		6×4 ×⅝	14×¾	113.2	71.4
613.9		6×4 ×¾	14×¾	127.6	71.4
200.4	26×7⁄16	4×3 ×½		83.1	
233.4		4×3 ×⅝		93.1	
233.5		5×3½×½		93.1	
265.8		6×4 ×½		103.5	
274.5		5×3½×⅝		105.9	
314.8		6×4 ×⅝		118.7	

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsb

***† (Continued). Elements of Riveted Plate Girders**

Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
b- te,	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
7/16	6×4 ×¾		133.1		78.8
	5×3½×½	12×½	93.1	40.8	78.8
	5×3½×½	12×⅝	93.1	51.0	78.8
	6×4 ×½	14×½	103.5	47.6	78.8
	5×3½×⅝	12×⅝	105.9	51.0	78.8
	6×4 ×½	14×⅝	103.5	59.5	78.8
	6×4 ×⅝	14×⅝	118.7	59.5	78.8
	6×4 ×⅝	14×¾	118.7	71.4	78.8
	6×4 ×¾	14×¾	133.1	71.4	78.8
5/16	5×3½×⅜		70.3		56.3
	6×4 ×⅜		77.9		56.3
	5×3½×½		83.1		56.3
	6×4 ×½		93.5		56.3
	5×3½×⅝		95.9		56.3
	5×3½×⅜	12×⅜	70.3	30.6	56.3
	6×4 ×⅝		108.7		56.3
	5×3½×⅜	12×½	70.3	40.8	56.3
3/8	6×4 ×⅝		115.7		67.5
	5×3½×⅜	12×½	77.3	40.8	67.5
	6×4 ×⅜	14×⅜	84.9	35.7	67.5
	6×4 ×¾		130.1		67.5
	5×3½×½	12×½	90.1	40.8	67.5
	6×4 ×⅜	14×½	84.9	47.6	67.5
	6×4 ×⅞		144.5		67.5
	5×3½×½	12×⅝	90.1	51.0	67.5
	6×4 ×½	14×½	100.5	47.6	67.5
	5×3½×⅝	12×⅝	102.9	51.0	67.5
	6×4 ×½	14×⅝	100.5	59.5	67.5
	6×4 ×⅝	14×⅝	115.7	59.5	67.5
	6×4 ×⅝	14×¾	115.7	71.4	67.5
	6×4 ×¾	14×¾	130.1	71.4	67.5
7/16	5×3½×½		96.1		78.8
	6×4 ×½		106.5		78.8
	5×3½×⅝		108.9		78.8
	6×4 ×⅝		121.7		78.8
	6×4 ×¾		136.1		78.8
	5×3½×½	12×½	96.1	40.8	78.8
	6×4 ×⅞		150.5		78.8
	5×3½×½	12×⅝	96.1	51.0	78.8
	6×4 ×½	14×½	106.5	47.6	78.8
	5×3½×⅝	12×⅝	108.9	51.0	78.8
	6×4 ×½	14×⅝	106.5	59.5	78.8
	6×4 ×⅝	14×⅝	121.7	59.5	78.8

ompanion, 1915 Edition, Carnegie Steel Company, *Pittsburgh, Pa.*

of table, *see page 706.*

Table IV *† (Continued). Elements of Riveted Plate Gird

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
628.0	28×7/16	6×4 ×⅝	14×¾	121.7	71.4
676.2		6×4 ×¾	14×¾	136.1	71.4
221.8	30×⅜	5×3½×⅜		79.9	
250.5		6×4 ×⅜		87.5	
272.1		5×3½×½		92.7	
310.3		6×4 ×½		103.1	
320.5		5×3½×⅝		105.5	
353.8		5×3½×⅜	12×⅜	79.9	30.6
366.2		5×3½×¾		117.5	
368.1		6×4 ×⅝		118.3	
397.8		5×3½×⅜	12×½	79.9	40.8
404.7		6×4 ×⅜	14×⅜	87.5	35.7
423.1		6×4 ×¾		132.7	
446.6		5×3½×½	12×½	92.7	40.8
456.1		6×4 ×⅜	14×½	87.5	47.6
475.8		6×4 ×⅞		147.1	
490.3		5×3½×½	12×⅝	92.7	51.0
514.0		6×4 ×½	14×½	103.1	47.6
536.7		5×3½×⅝	12×⅝	105.5	51.0
565.1		6×4 ×½	14×⅝	103.1	59.5
620.6		6×4 ×⅝	14×⅝	118.3	59.5
671.3		6×4 ×⅝	14×¾	118.3	71.4
723.8		6×4 ×¾	14×¾	132.7	71.4
281.4	30×7/16	5×3½×½		99.0	
319.5		6×4 ×½		109.4	
329.7		5×3½×⅝		111.8	
375.5		5×3½×¾		123.8	
377.3		6×4 ×⅝		124.6	
432.3		6×4 ×¾		139.0	
455.5		5×3½×½	12×½	99.0	40.8
485.0		6×4 ×⅞		153.4	
499.2		5×3½×½	12×⅝	99.0	51.0
523.0		6×4 ×½	14×½	109.4	47.6
545.6		5×3½×⅝	12×⅝	111.8	51.0
574.0		6×4 ×½	14×⅝	109.4	59.5
629.5		6×4 ×⅝	14×⅝	124.6	59.5
680.1		6×4 ×⅝	14×¾	124.6	71.4
732.6		6×4 ×¾	14×¾	139.0	71.4
290.6	30×½	5×3½×½		105.4	
328.8		6×4 ×½		115.8	
338.9		5×3½×⅝		118.2	
384.7		5×3½×¾		130.2	
386.5		6×4 ×⅝		131.0	

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pitts

† For explanation of table, see page 706.

*† (Continued). **Elements of Riveted Plate Girders**

Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
eb- te, n	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
	6×4 ×¾		145.4		99.0
	5×3½×½	12×½	105.4	40.8	99.0
	6×4 ×⅞		159.8		99.0
	5×3½×½	12×⅝	105.4	51.0	99.0
	6×4 ×½	14×½	115.8	47.6	99.0
½	5×3½×⅝	12×⅝	118.2	51.0	99.0
	6×4 ×½	14×⅝	115.8	59.5	99.0
	6×4 ×⅝	14×⅝	131.0	59.5	99.0
	6×4 ×⅝	14×¾	131.0	71.4	99.0
	6×4 ×¾	14×¾	145.4	71.4	99.0
	5×3½×⅜		83.7		81.0
	6×4 ×⅜		91.3		81.0
⅜	5×3½×½		96.5		81.0
	6×6 ×⅜		101.7		121.5
	6×4 ×½		106.9		81.0
	6×6 ×½		124.3		135.0
	5×3½×¾		125.1		87.8
	6×4 ×⅝		125.9		87.8
	6×4 ×⅜	14×⅜	95.1	35.7	87.8
	6×6 ×⅝		142.7		135.0
	6×4 ×¾		140.3		87.8
	6×6 ×⅜	14×⅜	105.5	35.7	135.0
	5×3½×½	12×½	100.3	40.8	87.8
	6×4 ×⅜	14×½	95.1	47.6	87.8
	6×6 ×⅜	14×½	105.5	47.6	135.0
	6×4 ×⅞		154.7		87.8
	5×3½×½	12×⅝	100.3	51.0	87.8
⅜	6×4 ×½	14×½	110.7	47.6	87.8
	6×6 ×⅜	14×⅝	105.5	59.5	135.0
	5×3½×⅝	12×⅝	113.1	51.0	87.8
	6×6 ×½	14×½	124.3	47.6	135.0
	6×4 ×½	14×⅝	110.7	59.5	87.8
	6×6 ×½	14×⅝	124.3	59.5	135.0
	6×4 ×⅝	14×⅝	125.9	59.5	87.8
	6×6 ×½	14×¾	124.3	71.4	135.0
	6×6 ×⅝	14×⅝	142.7	59.5	135.0
	6×4 ×⅝	14×¾	125.9	71.4	87.8
	6×6 ×⅝	14×¾	142.7	71.4	135.0
	6×4 ×¾	14×¾	140.3	71.4	87.8
	5×3½×½		108.0		102.4
	6×4 ×½		118.4		102.4
7/16	5×3½×⅝		120.8		102.4
	6×6 ×½		132.0		157.5

ompanion, 1915 Edition, Carnegie Steel *Company, Pittsburgh*, Pa.

of table, see page 706.

Table IV * † (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
473.3	36×7/16	5×3½×¾		132.8	
475.7		6×4 ×⅝		133.6	
523.8		6×6 ×⅝		150.4	
543.5		6×4 ×¾		148.0	
567.2		5×3½×½	12×½	108.0	40.8
608.6		6×4 ×⅞		162.4	
619.7		5×3½×½	12×⅝	108.0	51.0
649.5		6×4 ×½	14×½	118.4	47.6
677.1		5×3½×⅝	12×⅝	120.8	51.0
687.3		6×6 ×½	14×½	132.0	47.6
710.8		6×4 ×½	14×⅝	118.4	59.5
748.4		6×6 ×½	14×⅝	132.0	59.5
779.5		6×4 ×⅝	14×⅝	133.6	59.5
809.5		6×6 ×½	14×¾	132.0	71.4
825.9		6×6 ×⅝	14×⅝	150.4	59.5
840.4		6×4 ×⅝	14×¾	133.6	71.4
886.6		6×6 ×⅝	14×¾	150.4	71.4
905.5		6×4 ×¾	14×¾	148.0	71.4
418.0	36×½	6×4 ×½		126.0	
456.9		6×6 ×½		139.6	
489.0		6×4 ×⅝		141.2	
537.1		6×6 ×⅝		158.0	
556.9		6×4 ×¾		155.6	
614.5		6×6 ×¾		176.0	
621.9		6×4 ×⅞		170.0	
662.5		6×4 ×½	14×½	126.0	47.6
689.2		6×6 ×⅞		193.6	
700.3		6×6 ×½	14×½	139.6	47.6
723.7		6×4 ×½	14×⅝	126.0	59.5
761.3		6×6 ×½	14×⅝	139.6	59.5
792.3		6×4 ×⅝	14×⅝	141.2	59.5
822.3		6×6 ×½	14×¾	139.6	71.4
838.8		6×6 ×⅝	14×⅝	158.0	59.5
853.2		6×4 ×⅝	14×¾	141.2	71.4
899.4		6×6 ×⅝	14×¾	158.0	71.4
918.3		6×4 ×¾	14×¾	155.6	71.4
973.7		6×6 ×¾	14×¾	176.0	71.4
1 039.4		6×4 ×¾	14×1	155.6	95.2
1 094.1		6×6 ×¾	14×1	176.0	95.2
1 101.1		6×4 ×⅞	14×1	170.0	95.2
1 164.9		6×6 ×⅞	14×1	193.6	95.2
444.7	36×⅝	6×4 ×½		141.3	
483.5		6×6 ×½		154.9	

* From Pocket Companion, 1915 Editi[illegible]el Company, Pittsbu

† For explanation of table, see page 7

(Continued). Elements of Riveted Plate Girders

Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
b- te, n	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
	6×4×⅜		156.5		146.3
	6×6×⅜		173.3		225.0
	6×4×¾		170.9		146.3
	6×6×¾		191.3		225.0
	6×4×⅞		185.3		146.3
	6×4×½	14×½	141.3	47.6	146.3
	6×6×⅞		208.9		225.0
	6×6×½	14×½	154.9	47.6	
	6×4×½	14×⅝	141.3	59.5	146.3
	6×6×½	14×⅝	154.9	59.5	225.0
	6×4×⅝	14×⅝	156.5	59.5	146.3
⅜	6×6×½	14×¾	154.9	71.4	225.0
	6×6×⅝	14×⅝	173.3	59.5	225.0
	6×4×⅝	14×¾	156.5	71.4	146.3
	6×6×⅝	14×¾	173.3	71.4	225.0
	6×4×¾	14×¾	170.9	71.4	146.3
	6×6×¾	14×¾	191.3	71.4	225.0
	6×6×⅝	14×1	173.3	95.2	225.0
	6×4×¾	14×1	170.9	95.2	146.3
	6×6×¾	14×1	191.3	95.2	225.0
	6×4×⅞	14×1	185.3	95.2	146.3
	6×6×⅞	14×1	208.9	95.2	225.0
	6×4×⅜		102.8		101.3
	6×6×⅜		113.2		157.5
	6×4×½		118.4		101.3
	6×6×½		132.0		157.5
	6×4×⅝		133.6		101.3
	6×4×⅜	14×⅜	102.8	35.7	101.3
	6×6×⅝		150.4		157.5
	6×4×⅜	16×⅜	102.8	40.8	101.3
	6×4×¾		148.0		101.3
	6×6×⅜	14×⅜	113.2	35.7	157.5
⅜	6×6×⅜	16×⅜	113.2	40.8	157.5
	6×4×⅜	14×½	102.8	47.6	101.3
	6×6×⅜	14×½	113.2	47.6	157.5
	6×6×¾		168.4		157.5
	6×4×⅞		162.4		101.3
	6×6×⅜	16×½	113.2	54.4	157.5
	6×4×½	14×½	118.4	47.6	101.3
	6×6×⅜	14×⅝	113.2	59.5	157.5
	6×4×½	16×½	118.4	54.4	101.3
	6×6×⅞		186.0		157.5
	6×6×½	14×½	132.0	47.6	157.5

ompanion, 1915 Edition, Carnegie Steel Company, *Pittsburgh, Pa.*

of table, *see page 706.*

Table IV*† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
835.5	42×⅜	6×4×½	14×⅝	118.4	59.5
855.2		6×6×½	16×½	132.0	54.4
884.2		6×6×½	14×⅝	132.0	59.5
917.3		6×4×⅝	14×⅝	133.6	59.5
937.3		6×6×½	16×⅝	132.0	68.0
955.7		6×6×½	14×¾	132.0	71.4
970.4		6×4×⅝	16×⅝	133.6	68.0
977.6		6×6×⅝	14×⅝	150.4	59.5
988.7		6×4×⅝	14×¾	133.6	71.4
1 030.8		6×6×⅝	16×⅝	150.4	68.0
1 048.6		6×6×⅝	14×¾	150.4	71.4
1 066.6		6×4×¾	14×¾	148.0	71.4
1 112.4		6×6×⅝	16×¾	150.4	81.6
1 130.4		6×4×¾	16×¾	148.0	81.6
1 138.5		6×6×¾	14×¾	168.4	71.4
1 194.1		6×6×⅝	16×⅞	150.4	95.2
1 202.3		6×6×¾	16×¾	168.4	81.6
1 283.5		6×6×¾	16×⅞	168.4	95.2
1 286.4		6×4×⅞	16×⅞	162.4	95.2
1 369.9		6×6×⅞	16×⅞	186.0	95.2
495.3	42×7/16	6×4×½		127.3	
545.4		6×6×½		140.9	
579.5		6×4×⅝		142.5	
641.6		6×6×⅝		159.3	
660.2		6×4×¾		156.9	
734.7		6×6×¾		177.3	
737.6		6×4×⅞		171.3	
781.5		6×4×½	14×½	127.3	47.6
824.0		6×4×½	16×½	127.3	54.4
824.6		6×6×⅞		194.9	
830.4		6×6×½	14×½	140.9	47.6
853.1		6×4×½	14×⅝	127.3	59.5
872.9		6×6×½	16×½	140.9	54.4
901.8		6×6×½	14×⅝	140.9	59.5
934.9		6×4×⅝	14×⅝	142.5	59.5
954.9		6×6×½	16×⅝	140.9	68.0
973.2		6×6×½	14×¾	140.9	71.4
988.1		6×4×⅝	16×⅝	142.5	68.0
995.3		6×6×⅝	14×⅝	159.3	59.5
1 006.2		6×4×⅝	14×¾	142.5	71.4
1 048.4		6×6×⅝	16×⅝	159.3	68.0
1 066.2		6×6×⅝	14×¾	159.3	71.4
1 084.1		6×4×¾	14×¾	156.9	71.4

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

† For explanation of table, see page 706.

ble IV*† (Continued). Elements of Riveted Plate Girders

Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
42×⅞	6×6×⅝	16×¾	159.3	81.6	183.8
	6×4×¾	16×¾	156.9	81.6	118.1
	6×6×¾	14×¾	177.3	71.4	183.8
	6×6×⅝	16×⅞	159.3	95.2	183.8
	6×6×¾	16×¾	177.3	81.6	183.8
	6×6×¾	16×⅞	177.3	95.2	183.8
	6×6×⅞	16×⅞	194.9	95.2	183.8
42×½	6×4×½		136.2		135.0
	6×6×½		149.8		210.0
	6×4×⅝		151.4		135.0
	6×6×⅝		168.2		210.0
	6×4×¾		165.8		135.0
	6×6×¾		186.2		210.0
	6×4×⅞		180.2		135.0
	6×4×½	14×½	136.2	47.6	135.0
	6×4×½	16×½	136.2	54.4	135.0
	6×6×⅞		203.8		210.0
	6×6×½	14×½	149.8	47.6	210.0
	6×4×½	14×⅝	136.2	59.5	135.0
	6×6×½	16×½	149.8	54.4	210.0
	6×6×½	14×⅝	149.8	59.5	210.0
	6×4×⅝	14×⅝	151.4	59.5	135.0
	6×6×½	16×⅝	149.8	68.0	210.0
	6×6×½	14×¾	149.8	71.4	210.0
	6×4×⅝	16×⅝	151.4	68.0	135.0
	6×6×⅝	14×⅝	168.2	59.5	210.0
	6×4×⅝	14×¾	151.4	71.4	135.0
	6×6×⅝	16×⅝	168.2	68.0	210.0
	6×6×⅝	14×¾	168.2	71.4	210.0
	6×4×¾	14×¾	165.8	71.4	135.0
	6×6×⅝	16×¾	168.2	81.6	210.0
	6×4×¾	16×¾	165.8	81.6	135.0
	6×6×¾	14×¾	186.2	71.4	210.0
	6×6×⅝	16×⅞	168.2	95.2	210.0
	6×6×¾	16×¾	186.2	81.6	210.0
	6×6×¾	16×⅞	186.2	95.2	210.0
	6×4×⅞	16×⅞	180.2	95.2	135.0
	6×6×⅞	16×⅞	203.8	95.2	210.0
48×⅜	6×4×⅜		110.4		121.5
	6×6×⅜		120.8		180.0
	6×4×½		126.0		121.5
	6×6×½		139.6		180.0
	6×4×⅝		141.2		121.5

ocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

lanation of table, see page 706.

Table IV *† (Continued). Elements of Riveted Plate Gird

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
714.4	48×3/8	6×4×3/8	14×3/8	110.4	35.7
741.3		6×6×5/8		158.0	
750.8		6×4×3/8	16×3/8	110.4	40.8
758.5		6×4×3/4		155.6	
759.5		6×6×3/8	14×3/8	120.8	35.7
795.9		6×6×3/8	16×3/8	120.8	40.8
797.0		6×4×3/8	14×1/2	110.4	47.6
841.9		6×6×3/8	14×1/2	120.8	47.6
848.3		6×4×7/8		170.0	
850.1		6×6×3/4		176.0	
890.4		6×6×3/8	16×1/2	120.8	54.4
895.5		6×4×1/2	14×1/2	126.0	47.6
924.3		6×6×3/8	14×5/8	120.8	59.5
944.0		6×4×1/2	16×1/2	126.0	54.4
955.2		6×6×7/8		193.6	
955.8		6×6×1/2	14×1/2	139.6	47.6
977.7		6×4×1/2	14×5/8	126.0	59.5
1 004.3		6×6×1/2	16×1/2	139.6	54.4
1 037.6		6×6×1/2	14×5/8	139.6	59.5
1 072.7		6×4×5/8	14×5/8	141.2	59.5
1 098.2		6×6×1/2	16×5/8	139.6	68.0
1 119.5		6×6×1/2	14×3/4	139.6	71.4
1 133.3		6×4×5/8	16×5/8	141.2	68.0
1 147.1		6×6×5/8	14×5/8	158.0	59.5
1 154.4		6×4×5/8	14×3/4	141.2	71.4
1 207.8		6×6×5/8	16×5/8	158.0	68.0
1 228.4		6×6×5/8	14×3/4	158.0	71.4
1 245.2		6×4×3/4	14×3/4	155.6	71.4
1 301.2		6×6×5/8	16×3/4	158.0	81.6
1 317.9		6×4×3/4	16×3/4	155.6	81.6
1 334.0		6×6×3/4	14×3/4	176.0	71.4
1 394.7		6×6×5/8	16×7/8	158.0	95.2
1 406.7		6×6×3/4	16×3/4	176.0	81.6
1 498.1		6×4×7/8	16×7/8	170.0	95.2
1 499.7		6×6×3/4	16×7/8	176.0	95.2
1 601.3		6×6×7/8	16×7/8	193.6	95.2
591.2	48×7/16	6×4×1/2		136.2	
652.7		6×6×1/2		149.8	
688.7		6×4×5/8		151.4	
765.0		6×6×5/8		168.2	
782.3		6×4×3/4		165.8	
872.1		6×4×7/8		180.2	
873.8		6×6×3/4		186.2	

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* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsbur

† For explanation of table, see page 706.

ble IV*† (Continued). Elements of Riveted Plate Girders

Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
48×7/16	6×4×½	14×½	136.2	47.6	141.8
	6×4×½	16×½	136.2	54.4	141.8
	6×6×⅞		203.8		210.0
	6×6×½	14×½	149.8	47.6	210.0
	6×4×½	14×⅝	136.2	59.5	141.8
	6×6×½	16×½	149.8	54.4	210.0
	6×6×½	14×⅝	149.8	59.5	210.0
	6×4×⅝	14×⅝	151.4	59.5	141.8
	6×6×½	16×⅝	149.8	68.0	210.0
	6×6×½	14×¾	149.8	71.4	210.0
	6×4×⅝	16×⅝	151.4	68.0	141.8
	6×6×⅝	14×⅝	168.2	59.5	210.0
	6×4×⅝	14×¾	151.4	71.4	141.8
	6×6×⅝	16×⅝	168.2	68.0	210.0
	6×6×⅝	14×¾	168.2	71.4	210.0
	6×4×¾	14×¾	165.8	71.4	141.8
	6×6×⅝	16×¾	168.2	81.6	210.0
	6×4×¾	16×¾	165.8	81.6	141.8
	6×6×¾	14×¾	186.2	71.4	210.0
	6×6×⅝	16×⅞	168.2	95.2	210.0
	6×6×¾	16×¾	186.2	81.6	210.0
	6×4×⅞	16×⅞	180.2	95.2	141.8
	6×6×¾	16×⅞	186.2	95.2	210.0
	6×6×⅞	16×⅞	203.8	95.2	210.0
48×½	6×4×½		146.4		162.0
	6×6×½		160.0		240.0
	6×4×⅝		161.6		162.0
	6×6×⅝		178.4		240.0
	6×4×¾		176.0		162.0
	6×4×⅞		190.4		162.0
	6×6×¾		196.4		240.0
	6×4×½	14×½	146.4	47.6	162.0
	6×4×½	16×½	146.4	54.4	162.0
	6×6×½	14×½	160.0	47.6	240.0
	6×6×⅞		214.0		240.0
	6×4×½	14×⅝	146.4	59.5	162.0
	6×6×½	16×½	160.0	54.4	240.0
	6×6×½	14×⅝	160.0	59.5	240.0
	6×4×⅝	14×⅝	161.6	59.5	162.0
	6×6×½	16×⅝	160.0	68.0	240.0
	6×6×½	14×¾	160.0	71.4	240.0
	6×4×⅝	16×⅝	161.6	68.0	162.0
	6×6×⅝	14×⅝	178.4	59.5	240.0

ket Companion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.
nation of table, see *page 706.*

Table IV *† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
1 200.5	48×½	6×4×⅝	14×¾	161.6	71.4
1 254.1		6×6×⅝	16×⅝	178.4	68.0
1 274.5		6×6×⅝	14×¾	178.4	71.4
1 291.2		6×4×¾	14×¾	176.0	71.4
1 347.3		6×6×⅝	16×¾	178.4	81.6
1 364.0		6×4×¾	16×¾	176.0	81.6
1 380.0		6×6×¾	14×¾	196.4	71.4
1 440.6		6×6×⅝	16×⅞	178.4	95.2
1 452.8		6×6×¾	16×¾	196.4	81.6
1 543.9		6×4×⅞	16×⅞	190.4	95.2
1 545.6		6×6×¾	16×⅞	196.4	95.2
1 647.1		6×6×⅞	16×⅞	214.0	95.2

* From Pocket Companion, 1915 Edition, Carnegie Steel Company, Pittsburg

† For explanation of table, see page 706.

blems Stated. The problems which are presented in this part of istruction are, in general, (1) the designing of the joists and girders framework of the floor to safely support the greatest load likely to it, and (2) the determination of the maximum safe load for a floor lt. The first of these problems is the one with which architects s more commonly have to deal, and is, therefore, considered first.

f the Floor-Framing. Before any calculations can be made for the timbers it is necessary to know the spans of the joists, and, if enings in the floor, or the floor-joists have to support longitudinal a framing-plan should be made, showing the floor-area that will d by each joist, and also the position of partitions or special loads. is to be supported by posts and girders the position of these should rately indicated on the framing-plan.* Where the joists are supely by walls or partitions, the spans of the joists will of course be plan of the building. When the distance between a wall and a too great for a single span, there may be a question as to the best the posts and girders. When planning a building in which wooden be used, it is important to keep in mind the general scheme of the g and particularly the spans. Whenever practicable the spans of ts should not exceed 24 ft. When the distance between the sups exceeds 30 ft, girders should be placed so that the maximum span will not exceed 24 ft for light buildings nor from 16 to 18 ft for

Buildings it is desirable to have the rooms at least 27 ft wide, and s class of buildings the joists usually have spans of from 27 to 30 ft. f 30 ft, however, 16-in joists should be used, and as these are expenten difficult to obtain, it is much better and more economical to hoolrooms 27 by 32 or 34 ft, than to make them 30 ft square. A 27 ft wide by from 32 to 34 ft long, with windows on the long side, nomical and satisfactory, as it permits of using 3 by 14-in joists, nd also results in the most satisfactory lighting.

next step involves the determination of the loads for which the joists a
are to be proportioned. Floor-loads are made up of two parts, the
materials composing the floor itself, and the ceiling below, if there is
the load liable to be put on the floor. The first is called the DEAD L
the second the LIVE LOAD. When the SAFE LOAD for a floor is spok
live load is generally meant.

Weight of Wooden Floor-Construction. Wooden floors usually c
(1) beams, commonly called JOISTS,* or FLOOR-JOISTS, (2) one or two th
of flooring-boards, and, in a finished building, (3) a ceiling undern
joists. In figuring the weight of ⅞-in flooring-boards it will be su
accurate to estimate the weight of a single thickness at 3 lb per sq
joists may also be figured at 3 lb per ft, board-measure, with the exc
hard-pine and oak joists, which should be figured at 4 lb per ft, board-
The weight of the joists must also be reduced to their equivalent w
square foot of floor. Thus, the weight of a 2 by 12-in joist is about 6
ft. If the joists are spaced 12 in on centers, this will be equal to 6 lb
but if the joists are 16 in on centers there will be but one lineal foo
to every 1⅓ sq ft, which will be equivalent to 4½ lb per sq ft; and i
20 in on centers, the weight will be equal to 3⅗ lb per sq ft; spaced
centers, the weight will be 3 lb per sq ft. The weight of a lath-a
ceiling should be taken at 10 lb per sq ft, and of a ¾-in wooden ceilin
per sq ft. A corrugated-iron ceiling weighs about 1 lb per sq ft. For
steel ceilings, 2 lb per sq ft will cover the weight of the metal and furr
following table, giving the weight of joists, will be found convenient i
the weight of floors:

Table I. Weight of Floor-Joists per Square Foot of Floor

Sizes of joists	Spruce, hemlock, white pine		Hard pine
	Spacing in inches, center to center		Spacing in i center to c
	12	16	12
in	lb	lb	lb
2× 6	3	2¼	4
2× 8	4	3	5⅓
3× 8	6	4½	8
2×10	5	3¾	6⅔
3×10	7½	5½	10
2×12	6	4½	8
3×12	9	6¾	12
2×14	7	5¼	9⅓
3×14	10½	8½	14

Weight of Crowds. L. J. Johnson reports† results of some tests
tain the weight of crowds of men, in which he obtained weights of 13

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* Some building laws use the term FLOOR-BEAM instead of the word J

† See Engineering News, April 14, 1904.

.9 lb per sq ft. The last-mentioned weight was obtained by pack- a room about 6 by 11 ft in size. Professor Johnson also found nen in the room, making a load of 122 lb per sq ft, the crowd was o that a man could elbow his way through it only with persever- rmined effort."

sed Loads. There is much difference of opinion as to what uld be made for the live load. Table II shows the minimum live loads for different classes of buildings, as fixed by the build- e cities mentioned. (See, also, page 149.)

Minimum Safe Superimposed Loads for Floors, Required by Various Building Laws

f buildings	Minimum live load per square foot of floor					
	Buffalo, 1905	Boston, 1912	Chicago, 1911	Philadelphia, 1913	New York, 1906	St. Louis, 1910
....	40	50	40	70	60	60
nents and lodg-	70	50	50	70	60	60
ıgs	70	100	50	100	75 *	70 *
public assembly	100	200	100	120	90	100 †
ıouses and mfg.	120 ‡	125 ‡	100 ‡	120 ‡	120 ‡	150 ‡

t floor, 150 lb. † Also schoolhouses. ‡ And upwards.

ıpinion of Mr. Kidder that the following allowances for floor-loads, ction with the values given for the safe strength of joists or beams, ıte safety with proper allowance for economy.

	Lb per sq ft
ings, sleeping-rooms and lodging-rooms	40
lrooms	50
-buildings, upper stories	60
-buildings, first story	80
s and carriage-houses	65
ng-rooms, churches and theaters	80
ıbly-halls, dancing-halls and the corridors of all public rs, including hotels	120
ooms	150

for Stores and Buildings for Light Manufacturing. Floors for s, light manufacturing and light storage should be computed for 120 lb per sq ft, and for a concentrated load at any point of

ır Dwellings, etc. Floors of dwellings, tenements, lodging-houses ıotels, are seldom loaded with more than 20 lb per sq ft for the d a minimum load of 40 lb per sq ft should *provide for all* gencies.

more heavily loaded than others, it would not appear to be
average to determine the strength of floor-beams and floor-ar
would probably answer for the columns. There seems to be a c
ence of opinion among the leading architects and structural en
what allowance should be made for office-floors. Among so
fire-proof office-buildings, for example, may be mentioned t
Building in San Francisco in which the live loads were assum
ft for all floors above the first. In the Venetian Building, Chi
third and fourth floors were calculated for 60, and the upper f
sq ft of live load, while in the Old Colony and Fort Dearb
Chicago, the live loads on the floor-beams were assumed at
At the present time (1915), 50, 60, 70, 75, 100 and 150 lb
minimum live loads for the design of floors of office-buildings
building laws of six different cities. C. C. Schneider recom
design of floors of office-buildings above the first floor, for t
of the floor-area, 50; for concentrated loads applied at any p
5 000; and for the uniform load for girders, 1 000; the 50 bei
sq ft, the 5 000 in pounds and the 1 000 in pounds per linear fo

Live Loads for Churches, Theaters and School-Houses. "
120 lb per sq ft for the live load in churches, theaters and schoo
greater than the actual conditions require. The average size
is about 28 by 32 ft, and such a room usually contains seats for
and the teacher. Assuming the average weight of each schol
average live load, including ten visiting adults and the desks
13 lb per sq ft. Even supposing that the scholars of two ro
for some special occasion, there would be only 22 lb per sq ft
great a load as it is possible to imagine in such a room, as the fix
the crowding together of the scholars except at the sides of t
this reasoning, therefore, 50 lb per sq ft would appear ample
As a matter of fact, 3 by 14-in long-leaf yellow-pine joists,
and with a 28-ft span, have been used for school-room floors
such beams, if calculated by the formula for stiffness, would
load of only 43 lb per sq ft. (Table XII, page 643 and Tab

ge Floor-Loads for Stores has also been greatly over-estimated. ney found that the average load on the floors of the wholesale ware- rshall Field & Company, in Chicago, was but 50 lb per sq ft, and il stores will average over 80 lb per sq ft. An allowance of 120 lb ifficient for ordinary retail stores, with the possible exception of res.

s for Warehouses. Warehouses, on the other hand, may be very d, and the floors in buildings intended for the storage of merchan- e proportioned to the especial class of goods which they are de- pport. Table III, originally compiled by C. J. H. Woodbury,* some additions have been made by the Insurance Engineering tation and by Mr. Kidder, will be found of assistance in deciding load to be assumed for warehouse-floors. The weights per square single packages. If the goods are piled two or more cases high, r square foot of floor will of course be increased accordingly. In ht to which the goods are liable to be piled is a very important con- fixing upon the floor-load. In Table III "the measurements were to the outside of case or package, and gross weights of such pack- ."

of Determining the Sizes of Joists, Beams or Girders Re- ny Building. As already explained, the first step is the making plan of the floors or enough of it to show any special framing span and floor-area supported by the different joists, beams

Table III. Weights of Merchandise

Materials	Measurements		Weights		
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft	Per cu ft
	Wool				
ndia	3.0	12.0	340	113	28
lia	5.8	26.0	385	66	15
America	7.0	34.0	1 000	143	29
	6.9	33.0	482	70	15
nia	7.5	33.0	550	73	17
	5.0	30.0	200	40	7
ired wool	......	......	......	......	5
	Woollen Goods				
s	5.5	12.7	220	40	17
s, heavy	7.1	15.2	330	46	22
oods	5.5	22.0	460	84	21
eres	10.5	28.0	550	52	20
vear	7.3	21.0	350	48	16
ts	10.3	35.0	450	44	13
lankets	4.0	14.0	250	63	18

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* The Fire Protection *of Mills, page 118.*

Table III (Continued). Weights of Merchandise

Materials	Measurements		Weights	
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft
Cotton, Etc				
Bale	8.1	44.2	515	64
Bale, compressed	4.1	21.6	550	134
Bale, American Cotton Co.	4.0	11.0	263	66
Bale, Planters' Compressed Co.	2.3	7.2	254	110
Bale, jute	2.4	9.9	300	125
Bale, jute lashings	2.6	10.5	450	172
Bale, manila	3.2	10.9	280	88
Bale, hemp	8.7	34.7	700	81
Bale, sisal	5.3	17.0	400	75
Cotton Goods				
Bale, unbleached jeans	4.0	12.5	300	72
Piece duck	1.1	2.3	75	68
Bale, brown sheetings	3.6	10.1	235	65
Case, bleached sheetings	4.8	11.4	330	69
Case, quilts	7.2	19.0	295	41
Bale, print cloth	4.0	9.3	175	44
Case, prints	4.5	13.4	420	93
Bale, tickings	3.3	8.8	325	99
Skeins, cotton yarn	......	......	......	......
Burlaps	......	......	130	......
Jute bagging	1.4	5.3	100	70
Rags in Bales				
White linen	8.5	39.5	910	107
White cotton	9.2	40.0	715	78
Brown cotton	7.6	30.0	442	59
Paper shavings	7.5	34.0	507	68
Sacking	16.0	65.0	450	28
Woollen	7.5	30.0	600	80
Jute butts	2.8	11.1	400	143
Paper				
Calendered book	......	......	......	......
Supercalendered book	......	......	......	......
Newspaper	......	......	......	......
Strawboard	......	......	......	......
Leather-board	......	......	......	......
Writing	......	......	......	......
Wrapping	......	......	......	......
Manila	......	......	......	......

Table III (Continued). Weights of Merchandise

Materials	Measurements		Weights		
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft	Per cu ft
GRAIN *					
bags	4.2	4.2	165	39	39
ulk	......	......	......	......	44
ulk	......	......	......	......	39
ulk ...... mean	......	......	......	......	41
ur, on side	4.1	5.4	218	53	40
ur, on end	3.1	7.1	218	70	31
gs	3.6	3.6	112	31	31
in barrels	3.7	5.9	218	59	37
gs	3.3	3.6	96	29	27
	5.0	20.0	284	57	14
rick, compressed	1.75	5.25	125	72	24
erick, compressed	1.75	5.25	100	57	19
rick, compressed	1.75	5.25	150	86	29
Dederick, compressed	1.75	5.25	100	57	19
	......	......	......	......	4
DYESTUFFS, ETC					
bleaching powder	11.8	39.2	1 200	102	31
soda-ash	10.8	29.2	1 800	167	62
	3.0	9.0	385	128	43
	4.0	3.3	150	38	45
	1.6	4.1	160	100	39
a in iron drum	4.3	6.8	600	140	88
ch	3.0	10.5	250	83	23
l-alum	3.0	10.5	350	117	33
t logwood	1.06	0.8	55	52	70
	3.6	4.5	225	63	50
ent, American	3.8	5.5	325	86	59
ent, English	3.8	5.5	400	105	73
ter	3.7	6.1	325	88	53
n	3.0	9.0	430	143	48
-oil	4.3	12.3	422	98	34
	......	......	......	......	42
MISCELLANEOUS					
	2.7	0.5	139	99	278
	......	......	......	......	60
ery	9.9	39.6	1 600	162	40
ery	13.4	42.5	600	52	14
r	7.3	12.2	190	26	16
ins	11.2	16.7	300	27	18
des	6.0	30.0	400	67	13
des, compressed	6.0	30.0	700	117	23
ather	12.6	8.9	200	22	16
ther	......	......	......	......	17
ulated sugar	3.0	7.5	317	106	42
n sugar	3.0	7.5	340	113	45
	......	......	......	......	30

P. Cement 3¾ cu ft

re of grain in deep bins, see Engineering News, March 10, 1904, pages 224 Dec. 15., 1904.

The second step is to determine approximately the weight of the floor
ing, and decide what superimposed load per square foot the floor is to be
to carry. Having done this, the next step is the computing of the
dimensions of the common floor-joists. For most buildings the size
joists required can be readily determined by reference to Tables XIII
inclusive, and XXII to XXVI, inclusive, of this chapter. For other
the sizes of the common joists may be determined by computing th
be supported by a single joist and then, by the formulas or tables in
XVI or the formulas in Chapter XVIII, determining the dimensions of
to support that load. (See Example 1.) For the floors of all buildin
stores and warehouses it is recommended that the sizes of the comm
be determined by the formulas for stiffness in Chapter XVIII or the
values in the tables in Chapter XVI, unless one value, only, is given
for safe loads, in which case that value may be used. For stores a
houses the sizes of the joists may be proportioned by the formulas or
values of the tables in Chapter XVI.

The Dimensions of Special Beams, such as headers, trimmers and be
porting partitions, and also of the girders, should be determined in t
way, that is, by
ing the maxim
the beam may
support, and t
dimensions of
that will supp
load with safe
manner of ma
computation
plained in the
examples.

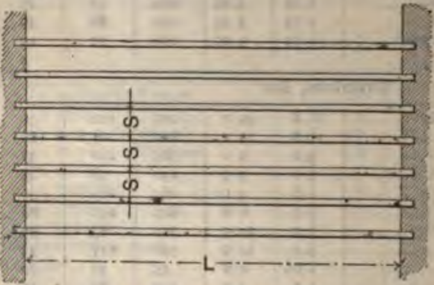


Fig. 1. Plan of Floor-joists

Example 1.
plest type of flo
ing is that show
1, in which al
joists are of t
span and support equal floor-areas. In such a floor, the FLOOR-AR
ported by each joist is equal to the span, L, multiplied by the spaci
feet. The LOAD on each joist is equal to the FLOOR-AREA multiplie
sum of the dead-loads and superimposed or live loads. To show the
tion of the above-mentioned formulas and tables we will assume tha
represents the framing of a floor in a dwelling-house or lodging-hou
$L = 18$ ft, $S = 16$ in or $1\frac{1}{3}$ ft, and that the timber is common white pi
joists are to support a plastered ceiling and a double floor of ⅞-in
What should be the size of the joists?

Solution. The FLOOR-AREA supported by each joist is $1\frac{1}{3}$ by 18, or
From Table XIII or XXII, pages 737 and 742, for a span of 18 ft, t
will probably have to be at least 2 by 12 in, and their weight will be abo
per sq ft (see Table I, page 718). The plastered ceiling weighs about
the flooring 6 lb per sq ft, making the total weight of the floor $20\frac{1}{2}$ lb
For the superimposed load we should allow at least 40 lb per sq ft (
719). This might be greater, if exacted by any particular building la
load on a single joist will, therefore, be, with these assumed unit loads
by 24 sq ft, or 1 452 lb.

From Table VIII, page 639, we find that the maximum load for

pine joist of 18 ft span is 623 lb; hence to support 1 452 lb will
eadth equal to $1452/623 = 2\frac{1}{3}$ in. Therefore, to comply with the
s for both strength and stiffness, the joists should be $2\frac{1}{3}$ by 12 in.
t a stock size. Joists 2 by 12 in, 12 in on centers, may next be
n joist must support 1 116 lb, requiring, by Table VIII, page
y 12-in joist, determined by the quotient 1 116/623. So that, in
, white-pine joists of a nominal size of 2 by 12 in and spaced 12 in
ight be used, although they are slightly under the required depth,
ed size is about $1\frac{3}{4}$ by $11\frac{1}{2}$ in. From Table VI, page 637, the
actor is 1.61, and $623 \text{ lb} \times 1.61 = 1\,003$ lb which is less than 1 116 lb,
be supported. From Tables XIII and XXII, pages 737 and 742,
n spans for 2 by 12 in white-pine joists, 12 in on centers, are 19 ft
in respectively, according to the assumed value of the modulus of
white pine. For 3 by 12-in joists, 16 in on centers, the load is
d $1\,506/623 = 2\frac{2}{5}$ in. The dressed size is almost $2\frac{3}{4}$ by $11\frac{1}{2}$ in,
on-factor, 2.53, and $623 \times 2.53 = 1\,576$ lb, an amount greater than
ables XIII
again, give
and 19 ft
maximum
ts 3 by 12,
nters, are
an neces-
this ex-
an is made
able VIII,
for 12-in
ues for the
re found,
ller, stiff-
hould be
the deflec-
t be con-

. Fig. 2
al section
, in which
oor joists
plastered
ich also
ttic joists.
l be the
cond-floor
t the re-
f STRENGTH, the timber being fair-quality Eastern spruce with a
ss assumed to be 700 lb per sq in for flexure? As the effect of a
load, compared with a distributed load, in producing deflection, is
s the comparative effect in producing rupture, whenever a beam
rable CONCENTRATED load it may be calculated by the formula or
RENGTH ONLY.

The first step will be to determine the load on a single floor-joist.
ne, as a trial, that the joists are to be 2 by 10 in, 12 in on centers,
first-story and second-story ceilings are to be *plastered, and that*

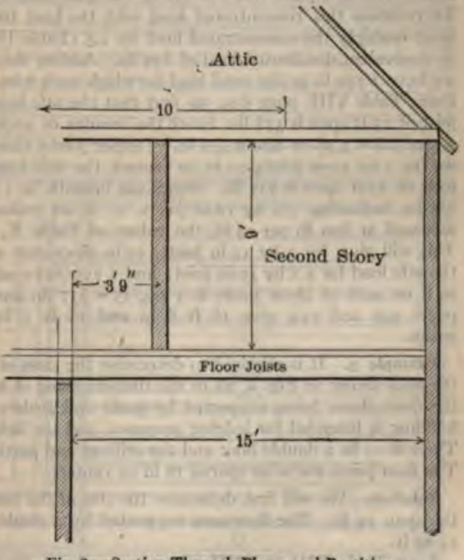
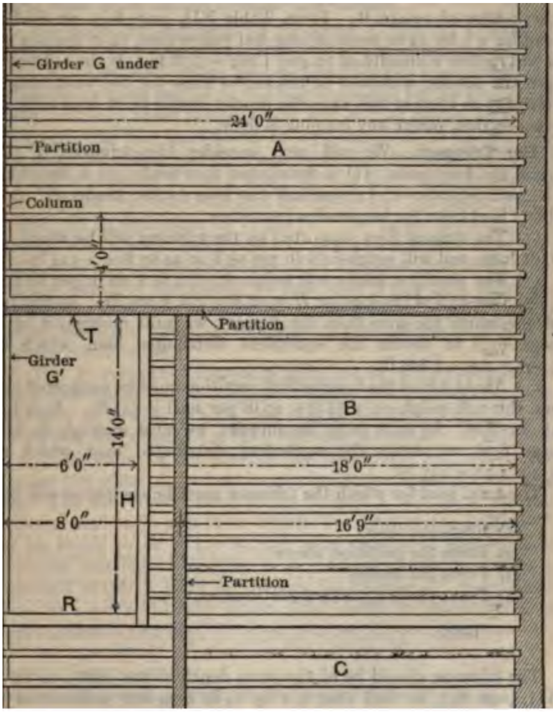


Fig. 2. Section Through Floors and Partitions

load from the partition and attic-floor. The attic-floor and c
16 lb per sq ft, and 24 lb is a sufficient allowance for the live
per linear foot on the partition will therefore be 400 lb. A
4-in studs, lathed and plastered on both sides, weighs about
face; hence the partition itself weighs 180 lb per lin ft.
attic-floor, therefore, bring a load of 580 lb on each secon
CENTRATED at a point ONE-FOURTH of the span from the inne
To combine this concentrated load with the load from the
must multiply the concentrated load by 1.5 (Table IV, page
an equivalent distributed load of 870 lb. Adding this to th
we have 1 740 lb as the total load for which each joist shoul
From Table VIII, page 639, we find that the safe load for a
joist of 15-ft span is 518 lb; hence the breadth of each joist
1 740/518 = 3.36 or about 3⅓ in. Deeper joists, therefore,
we try 2 by 12-in joists, 12 in on centers, the safe load for a
joist of 15-ft span is 747 lb. Hence the breadth is 1 755/74
2⅓ in, indicating 2⅓ by 12-in joists, 12 in on centers. If
assumed at 800 lb per sq in, the values of Table X, page
This will give, for 2 by 12-in joists, 12 in on centers and 15-
the safe load for a 1 by 12-in joist; and 1 755/850 = about 2
sq ft on each of these joists is 1 755/15 = 117 lb; and Tabl
pages 739 and 744, give 16 ft 6 in and 16 ft 1 in for t
spans.

Example 3. It is required to determine the sizes of the gi
the floor shown in Fig. 3, all of the timbers being of long-lea
the floor above being supported by posts and girders in the
building is intended for lodging purposes, and the height of
There is to be a double floor and the ceilings and partitions a
The floor-joists are to be spaced 16 in on centers.

Solution. We will first determine the size of the common
the span 24 ft. The floor-area supported by a single joist i
32 sq ft.


Girder G under
24′ 0″
Partition
A
Column
4′ 0″
T
Partition
Girder G′
14′ 0″
B
6′ 0″
18′ 0″
H
8′ 0″
16′ 9″
Partition
R
C

weight will be supported by the header and one-ninth by the w
of 2 532 is 2 251 lb, which, added to the floor-load, makes
the header of 10 410 lb. From Table XII, page 643, we f
load for a 1 by 14-in beam of long-leaf yellow pine, 14-ft span,
it will require a breadth of 10 410/1 867 = 5.58 in. If the tail-
into the header, it should be thicker to allow for the weaken
framing; so that, in this case, the header should be at least 6
cross-section, before any framing is done.

The Trimmers. We will next consider the trimmer, T
beam has four loads: (1) a distributed floor-load; (2) a dist
the partition above; (3) one-half the load on the header *H*;
direct load from the longitudinal partition.

(1) The strip of floor supported by the trimmer will be abo
24 ft long, and will weigh 64¾ lb per sq ft × 24 sq ft = 1 554 l

(2) The partition above will weigh 10 × 24 ft × 20 lb per s

(3) One-half of the load on *H* is 10 410/2 = 5 205 lb. As th
at one-fourth the span from the support, we must multiply it
page 632) to obtain the equivalent distributed load, whic
5 205 × 1.5 = 7 808 lb.

(4) About 8 in of the longitudinal partition must be supporte
and this will weigh 10 × ⅔ ft × 20 lb per sq ft = 133 lb. As i
at one-third the span from the support, we must multiply b
page 632) to obtain the equivalent distributed load, whic
133 × 1.78 = 237 lb.

The total load for which the trimmer must be computed wil

(1) From the floor. . . . .
(2) From the partition above. . . . .
(3) From the header. . . . .
(4) From the longitudinal partition. . . . .

Total. . . . .

The trimmer should be of the same depth as the joists, 14

in is taken for S, Table XV, page 646, is used, giving 14 399/
or the width. Hence, the architect will be governed by laws
ngineering judgment or experience elsewhere, and this applies
ell as to the girders. If wooden trimmers are used, they should
hangers (see last part of this chapter). The load on the trim-
he same as on the trimmer, T, except for the cross-partition.
eight of this partition, we have $14\,399 - 4\,800 = 9\,599$ lb for
stributed load on R, which, from Table XII, page 643, gives, for
dth 10.88 in or 8.8 in, depending upon whether the deflection
idered. Other variations in the required width of a 14-in
ll result from the use of other fiber-stresses.

The floor-area supported by the girder, G (Fig. 3), is equal to
8 sq ft. As a general rule, it will be safe in estimating the live
o take only 85% of the load assumed for the floor-beams, be-
lways be some portion of the floor supported by the girder that
d probably other portions that will not be loaded up to the
Hence, the live load would be 85% of 40 lb, or 34 lb. The
floor and ceiling will be about 25 lb, and the girder itself will
and 2 lb per sq ft, say 2 lb per sq ft of floor, more, so that we
sq ft for the total floor-load on this girder. As girder G sup-
his will be equivalent to 17 568 lb. The girder supports, also,
igh, above, which will weigh $12 \times 9 \times 20 = 2\,160$ lb. The total
e girder should be proportioned is, therefore, 19 728 lb. As-
the depth of the girder, we find from Table XII, page 643,
d for a 1 by 14-in long-leaf yellow-pine beam of 12-ft span is
he breadth of girder, G, should be $19\,728/1\,867 = 10.56$ in and
rder could be used.

(Fig. 3), supports a floor-area at the left of $12 \times 12 = 144$ sq ft,
a distributed load of 8 784 lb. On the right side of the girder
floor 40 in wide by 12 ft long (8 in of the floor being included
which will weigh 2 440 lb. This may be considered as a con-
pplied 20 in, or one-seventh the span, from the end of the
ase the effect of the load is practically the same as if the load
The load coming upon girder G' from T will equal one-half
uted load on T, plus three-eighths ($\frac{1}{2}$ of $\frac{3}{4}$) of the load on H.
we found to be 10 410 lb, and three-eighths of this is about
ctual distributed load on T we found to be $1\,554 + 4\,800 =$
e-half of this is 3 177 lb. Hence the trimmer, T, transmits a
$177 = 7\,077$ lb to the girder, which must be considered as a
l applied at one-third the span from the support, and hence we
by 1.78 (Table IV, page 632) to obtain the equivalent dis-
ich gives 12 597 lb.
hich the girder, G' (Fig. 3), should be computed will be

or at the left 8 784 lb

Baltimore, Buffalo, Milwaukee and New Haven, Table XV,
Philadelphia and Minneapolis, Table XII, page 643, with t
increased one-third. Of course, the sizes of the timbers are
assumed safe fiber-stresses are increased.

This example illustrates nearly all of the computations th
determine the sizes of the joists and special beams or girde
floor-construction. The method of computation is the sa
load, the only difference being that the greater the live load a
will be the loads for which the beams must be proportioned.
the most laborious computations are those for beams which
different sources, and it will generally be found that the w
any particular floor are the headers, trimmers and girders, an
support partitions.

The Strength of Mill-Floors. The beams and gird
should be computed by the same general method illustrate
examples, involving, (1) the determination of the loads o
girders and, (2) the sizes of the beams and girders require
loads.

Required Thickness of Plank Flooring. The thickness
ing in mill construction may be determined by formulas (1) a

$$\left.\begin{array}{r}\text{Thickness of plank in in}\\ \text{required for strength}\end{array}\right\} = \sqrt{\frac{\text{weight per sq ft}}{24 \times A}}$$

$$\left.\begin{array}{r}\text{Thickness of plank in in}\\ \text{required for stiffness}\end{array}\right\} = \sqrt[3]{\frac{\text{weight per sq ft}}{19.2 \times e_1}}$$

In these formulas, l is the span in feet, from center to cent
constants for strength (page 628), and e_1 the constant for stiff

When the planks are connected by $\frac{3}{4}$-in splines, and exten
Formula (1) may be used. If the planks are in single lengths fr
or are not splined, then Formula (2) should be used.

Tables IV to XI,* inclusive, show the safe loads for plank fl llow pine, and of Chicago and most other cities for Douglas fir.

V, E is 1 200 000 lb per sq in, S, 1 000 lb per sq in, and A is 56. given satisfy the requirements of Chicago and of most other cities for short-leaf yellow pine. The values given for stiffness, also, are ed for this wood.

VI, E is 1 200 000 lb per sq in, S, 800 lb per sq in, and A is 44. iven satisfy the requirements of all cities for strength, for spruce, ne and white pine, and the values given for stiffness, also, are ed for spruce. For Norway pine, E = 1 100 000 lb per sq in may

VII, E is 1 000 000 lb per sq in, S, 700 lb per sq in, and A is 39. given for strength can be used for any woods of that safe fiber- the loads for stiffness are recommended for white pine.

s VIII, IX, X and XI, the safe loads are calculated from still other , A, E and e_1, indicated with each table, and may be used by those o assume larger safe values for the strength and stiffness-factors re there are no restrictions from building laws. For any other values equired, such values must be inserted in Formula (1) or (2) and the of the planks determined or the safe load determined for any given planks.

Table IV. Safe Live Loads* in Pounds per Square Foot for Plank

See explanation on pages 730-1. The loads are based on the following
Strength: $S = 1\,200$ lb per sq in, $A = 67$; stiffness: $E = 1\,500\,000$ lb per sq

LONG-LEAF YELLOW PINE AND DOUGLAS FIR

Thickness of planks, in	Distance between centers of floor-beams, in feet						
	4	5	6	7	8	9	10
1⅝	353 229	226 117	157 68	115 43	88 29	70 20	....
2⅝	567 466	363 239	252 138	185 87	142 58	112 41	91 30
2¾	760 724	486 371	338 214	248 135	190 90	150 64	122 46
3½	....	788 764	547 442	402 278	308 187	243 131	197 95
4	....	....	715 660	525 416	402 278	318 196	257 143
5	....	....	....	820 812	628 544	496 382	402 278
6	....	....	....	....	904 940	715 660	579 481

* Weight of ceiling, if any, and also of the flooring itself is to be deducted
values.

Table V. Safe Live Loads* in Pounds per Square Foot for Plank

See explanation on pages 730-1. The loads are based on the following
Strength: $S = 1\,000$ lb per sq in, $A = 56$; stiffness: $E = 1\,200\,000$ lb per sq

SHORT-LEAF YELLOW PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet						
	4	5	6	7	8	9	10
1⅝	295 182	189 93	131 54	96 34	74 23	....	....
2⅝	474 370	303 189	211 110	155 69	118 46	94 32	76 24
2¾	635 574	406 294	282 170	207 107	159 72	125 50	102 37
3½	1 029	659 606	457 351	336 221	257 148	203 104	165 76
4	....	860	597 523	439 330	336 221	265 155	215 113
5	....	....	933	686 644	525 431	415 303	336 221
6	....	....	....	987	756 745	597 523	484 382

* Weight of ceiling, if any, and also of the flooring itself is to be deducted
values.

VI. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

explanation on pages 730-1. The loads are based on the following values.
th: $S = 800$ lb per sq in, $A = 44$; stiffness: $E = 1\ 200\ 000$ lb per sq in, $e_1 = 92$

SPRUCE, NORWAY PINE AND WHITE PINE

cness nks,	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
	232 182	148 93	103 54	76 34	58 23	....	....	....	....
	372 370	238 189	165 110	122 69	93 46	74 32	60 24	....	....
	499	319 294	222 170	163 107	125 72	99 50	80 37	66 28	....
	809	517	359 351	264 221	202 148	160 104	129 76	107 57	89 44
	....	676	469	345 330	264 221	209 155	169 113	140 85	117 65
	....	....	733	539	412	326 303	264 221	218 166	183 128
	....	....	....	776	594	469	380	314 287	264 221

ight of ceiling, if any, and also of the flooring itself is to be deducted from these

VII. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

e explanation on pages 730-1. The loads are based on the following values.
h: $S = 700$ lb per sq in, $A = 39$; stiffness: $E = 1\ 000\ 000$ lb per sq in, $e_1 = 77$

OR HEMLOCK AND WOODS OF SIMILAR STRENGTH AND STIFFNESS

cness nks,	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
	206 152	132 78	91 45	67 28	....	....	....	....	....
	330 309	212 158	147 92	108 58	82 39	65 27	....	....	....
	442 480	283 246	197 142	146 90	111 60	87 42	71 31	....	....
	717	459	319 293	234 185	179 124	142 87	115 63	95 48	80 37
	936	599	416	306 276	234 185	185 130	150 95	124 71	104 55
	....	936	650	478	366 361	289 253	234 185	193 139	163 107
	....	....	936	688	526	416	337 319	278 240	234 185

ight of ceiling, if any, and also of the flooring itself is to be *deducted from these*

Table VIII. Safe Live Loads* in Pounds per Square Foot for Plank F

See explanation on pages 730-1. The loads are based on the following va
Strength: $S = 1\ 800$ lb per sq in, $A = 100$; stiffness: $E = 1\ 780\ 000$ lb per sq in,
Recommended by Mr. Kidder for
LONG-LEAF YELLOW PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet							
	4	5	6	7	8	9	10	11
1⅞	515 258	325 126	222 68	160 38	120 21	92 11	72 5	....
2⅜	831 536	527 268	362 149	262 88	197 54	153 34	121 24	97 12
2¾	1 118 838	710 421	488 237	354 144	267 91	208 59	165 38	134 25
3½	....	1 158 884	798 504	582 310	442 202	345 136	276 94	225 67
4	....	....	1 046 759	763 470	580 308	454 210	364 148	296 106
5	....	....	....	1 200 934	913 618	716 427	576 304	471 223
6	....	....	....	....	1 322 1 081	1 038 751	836 540	686 398

* Weight of ceiling, if any, to be deducted. The weight of the flooring has bee
from values derived from formulas. Deduction about 72 lb per cu ft floor-

Table IX. Safe Live Loads* in Pounds per Square Inch for Plank F

See explanation on pages 730-1. The loads are based on the following v
Strength: $S = 1\ 620$ lb per sq in, $A = 90$; stiffness: $E = 1\ 425\ 000$ lb per sq i
Recommended by Mr. Kidder for
DOUGLAS FIR AND SHORT-LEAF YELLOW PINE

Thickness of planks, in	Distance center to center of floor-beams, in feet							
	4	5	6	7	8	9	10	11
1⅞	462 205	291 99	199 52	143 28	106 15	81 7	64	....
2⅜	747 428	473 212	324 117	234 68	176 41	136 25	107 14	.....
2¾	1 005 670	637 335	438 187	317 112	239 69	185 44	147 28	119 17
3½	....	1 040 706	717 401	522 246	395 159	308 106	246 72	200 50
4	....	1 362 1 061	940 606	685 374	520 244	406 165	325 115	265 81
5	....	....	1 476 1 198	1 078 745	819 491	642 338	516 240	422 174
6	....	....	....	1 560 1 302	1 187 863	932 597	749 428	614 314

* Weight of ceiling, if any, to be deducted. The weight of the flooring has bee
from values derived from formulas. Deduction about 72 lb per cu ft floor-

X. Safe Live Loads * in Pounds per Square Foot for Plank Flooring

xplanation on pages 730-1. The loads are based on the following values.

$S = 1\,260$ lb per sq in, $A = 70$; stiffness: $E = 1\,294\,000$ lb per sq in, $e_1 = 100$.

Recommended by Mr. Kidder for

SPRUCE

ness nks,	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
	360	227	155	111	83	64	50	.....	.....
	188	92	49	28	15	8	.....	.....	.....
	581	368	252	182	137	105	83	67	54
	391	194	108	64	39	24	15	.....	.....
	782	496	341	247	186	144	115	93	76
	612	307	173	104	66	42	28	18	.....
	1 228	781	548	391	296	231	184	150	124
	1 274	644	367	225	146	98	68	47	33
	.....	1 060	731	533	405	317	253	207	171
	.....	968	554	343	225	153	108	77	56
	.....	.....	1 148	839	638	500	402	329	273
	.....	.....	1 093	682	450	311	212	162	120
	.....	.....	.....	1 213	924	725	583	478	400
	.....	.....	.....	1 188	789	548	394	290	220

ht of ceiling, if any, to be deducted. The weight of the flooring has been deducted es derived from formulas. Deduction about 72 lb per cu ft floor-material.

XI. Safe Live Loads * in Pounds per Square Foot for Plank Flooring

xplanation on pages 730-1. The loads are based on the following values.

$S = 1\,080$ lb per sq in, $A = 60$; stiffness: $E = 1\,073\,000$ lb per sq in, $e_1 = 82$

Recommended by Mr. Kidder for

WHITE PINE

ness ks,	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
	307	193	131	94	70	53	41	.....	.....
	153	74	39	21	11	5	.....	.....	.....
	496	314	214	154	116	89	70	56	.....
	318	157	85	50	40	18	10	.....	.....
	668	424	290	210	158	122	97	78	63
	499	249	139	83	52	33	20	12	.....
	1 088	691	476	346	261	203	162	131	108
	1 041	526	298	183	119	78	53	36	25
	.....	906	625	455	345	269	215	175	145
	.....	791	451	278	181	123	85	60	43
	.....	.....	982	716	544	426	342	281	232
	.....	.....	893	555	366	251	178	129	95
	.....	.....	1 419	1 037	789	619	497	407	339
	.....	.....	1 553	970	643	445	319	234	175

t of ceiling, if any, to be deducted. The weight of the flooring *has been deducted* derived from formulas. *Deduction about 72 lb per cu ft floor-material.*

the size of a room and the purpose for which it is to be used
floor-joists required can be determined at a glance. Incide
show, also, the kind of wood most economical to use. If, o
being irregular in shape, the joists must be of different lengt
thickness of the joists may be varied, so that the same de
throughout.

Precautions Required in Using Tables. The precautions n
these tables are in regard to the superimposed loads and th
the timbers. The TOTAL LOADS for which the maximum span
puted are given at the head of each table. The actual w
(joists, flooring, plastering and deafening, if any) subtracted fr
will give the SUPERIMPOSED LOAD, that is, the load which the f
carry. If the ACTUAL SIZES of the joists are less than the NOM
the spans or spacings must be reduced from those given in t
the STOCK SIZES of joists generally run from ¼ in to ⅜ in scan
DIMENSIONS, this fact should always be taken into account
upon the sizes of joists. In this connection it will be conven
that 2-in joists, spaced 16 in on centers, have the same strengt
12 in on centers. A reduction should also be made for any
JOISTS that may be required. No allowance has been made fo
when they are to be supported by the floor-joists, additiona
used or the span reduced according to the relative direction
partitions and joists.

Tables XII to XX. Tables XII to XVI, inclusive, were
FORMULA FOR STIFFNESS (Chapter XVI, page 636 and Chapter
on the assumption that the deflection should not exceed $\frac{1}{30}$ in
They are based on the values of E (the modulus of elasticity)
F. E. Kidder. Tables XVII to XX, inclusive, were computed
FOR STRENGTH (Chapter XVI, page 635), and values for S (the
recommended by Mr. Kidder. The spans given in Tables X
sive, come within the requirements of the Buffalo and Denv
and Tables XII, XIV, XV, XVI and XVII comply with the
very nearly with the New York law; but to comply with th

Table XII. Maximum Span for Ceiling-Joists

See explanatory notes on page 736

Distance on centers	Hemlock, * $E=$ 1 045 000		White pine, $E=1\ 073\ 000$		Norway pine or spruce, $E=1\ 294\ 000$		Douglas fir or Texas pine, $E=1\ 425\ 000$		Long-leaf yellow pine, $E=1\ 780\ 000$	
in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 20 pounds per square foot										
12	9	3	9	5	10	1	10	5	11	2
16	8	5	8	6	9	1	9	5	10	1
12	14	0	14	1	15	1	15	7	16	8
16	12	8	12	10	13	8	14	2	15	2
12	18	8	18	10	20	1	20	9	22	4
16	17	0	17	2	18	4	18	11	20	5
20	15	9	15	10	17	0	17	6	18	10
Total load, 24 pounds per square foot										
12	22	0	22	2	23	8	24	5	26	4
16	20	0	20	2	21	7	22	3	23	10
20	18	6	18	8	20	0	20	7	22	2
12	26	5	26	8	28	5	29	4	31	7
16	24	0	24	2	25	10	26	8	28	8
20	22	3	22	5	24	0	24	8	26	8

* E is the modulus of elasticity and is in pounds per square inch.

XIII. Maximum Span for Floor-Joists for Dwellings, Tenements and Grammar-School Rooms with Fixed Desks

See explanatory notes on page 736

Distance on centers	Hemlock, * $E=$ 1 045 000		White pine, $E=1\ 073\ 000$		Norway pine or spruce, $E=1\ 294\ 000$		Douglas fir or Texas pine, $E=1\ 425\ 000$		Long-leaf yellow pine, $E=1\ 780\ 000$	
in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 60 pounds per square foot										
12	9	9	9	10	10	5	10	10	11	7
16	8	9	8	10	9	6	9	10	10	6
12	11	1	11	2	12	0	12	5	13	4
16	10	1	10	2	10	10	11	2	12	1
12	12	11	13	1	13	11	14	5	15	6
16	11	9	11	10	12	8	13	1	14	1
12	14	9	14	11	16	0	16	6	17	8
16	13	6	13	7	14	6	15	0	16	2
12	16	2	16	4	17	5	18	0	19	4
16	14	9	14	10	15	9	16	4	17	7
Total load, 66 pounds per square foot										
12	18	0	18	1	19	3	20	0	21	6
16	16	3	16	5	17	7	18	2	19	6
12	18	10	19	0	20	3	20	10	22	6
16	17	2	17	3	18	4	19	0	20	6
12	21	6	21	8	23	2	24	0	25	9
16	19	7	19	8	21	1	21	9	23	5
12	22	0	22	2	23	8	24	4	26	3
16	20	0	20	1	21	6	22	2	23	10
12	23	8	23	10	25	6	26	3	28	3
16	21	6	21	8	23	2	23	10	25	8
12	25	4	25	4	27	1	28	0	30	1
16	23	0	23	0	24	7	25	4	27	4

* E is the modulus of elasticity and is in pounds per square inch.

Table XIV. Maximum Span for Floor-Joists for Office-Buildings

See explanatory notes on page 736

Sizes of joists in	Distance on centers in	White pine, * E=1 073 000		Norway pine or spruce, E=1 294 000		Douglas fir or Texas pine, E=1 425 000		Long- yellow E=1	
		ft	in	ft	in	ft	in	ft	
Total load, 93 pounds per square foot									
3×8	12	12	10	13	9	14	2	15	[illegible]
3×8	16	11	8	12	6	12	10	13	[illegible]
2×10	12	14	1	15	1	15	6	16	[illegible]
2×10	16	12	9	13	8	14	1	15	[illegible]
3×10	12	16	1	17	3	17	9	19	[illegible]
3×10	16	14	8	15	8	16	2	17	[illegible]
2×12	12	16	10	18	1	18	8	20	[illegible]
2×12	16	15	4	16	5	17	0	18	[illegible]
Total load, 96 pounds per square foot									
3 ×12	12	19	2	20	6	21	2	22	[illegible]
3 ×12	16	17	5	18	7	19	3	20	[illegible]
2 ×14	12	19	6	20	10	21	7	23	[illegible]
2 ×14	16	17	9	19	0	19	7	21	[illegible]
2½×14	12	21	1	22	6	23	2	25	[illegible]
2½×14	16	19	2	20	4	21	2	22	[illegible]
3 ×14	12	22	4	23	10	24	8	27	[illegible]
3 ×14	16	20	4	21	8	22	5	24	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XV. Maximum Span for Floor-Joists for Churches and Theaters with Fixed Seats

See explanatory notes on page 736

Sizes of joists in	Distance on centers in	White pine, * E=1 073 000		Norway pine or spruce, E=1 294 000		Douglas fir or Texas pine, E=1 425 000		Long-leaf yellow E=1	
		ft	in	ft	in	ft	in	ft	in
Total load, 102 pounds per square foot									
3×8	12	12	6	13	4	13	9	14	[illegible]
3×8	16	11	4	12	2	12	6	13	[illegible]
2×10	12	13	7	14	7	15	1	16	[illegible]
2×10	16	12	4	13	3	13	8	14	[illegible]
3×10	12	15	8	16	9	17	3	18	[illegible]
3×10	16	14	2	15	2	15	8	16	[illegible]
2×12	12	16	5	17	7	18	1	19	[illegible]
2×12	16	14	10	15	11	16	5	17	[illegible]
Total load, 105 pounds per square foot									
3 ×12	12	18	7	19	11	20	6	22	[illegible]
3 ×12	16	16	10	18	1	18	7	20	[illegible]
2 ×14	12	19	0	20	3	20	10	22	[illegible]
2 ×14	16	17	3	18	5	19	0	20	[illegible]
2½×14	12	20	4	21	9	22	6	24	[illegible]
2½×14	16	18	7	19	10	20	6	22	[illegible]
3 ×14	12	21	8	23	2	23	10	25	[illegible]
3 ×14	16	19	8	21	1	21	9	23	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

XVI. Maximum Span for Floor-Joists for Assembly-Halls and Corridors

See explanatory notes on page 736

Distance on centers in	White pine, * $E = 1\ 073\ 000$ ft	in	Norway pine or spruce, $E = 1\ 294\ 000$ ft	in	Douglas fir or Texas pine, $E = 1\ 425\ 000$ ft	in	Long-leaf yellow pine, $E = 1\ 780\ 000$ ft	in
Total load, 123 pounds per square foot								
12	11	7	12	7	13	0	14	0
16	10	8	11	4	11	9	12	8
12	12	10	13	9	14	2	15	2
16	11	7	12	6	12	10	13	10
12	14	8	15	8	16	2	17	5
16	13	4	14	3	14	9	15	10
12	15	4	16	6	17	0	18	3
16	14	0	15	0	15	5	16	7
Total load, 126 pounds per square foot								
12	17	6	18	8	19	3	20	9
16	15	10	17	0	17	7	18	11
12	17	10	19	1	19	8	21	2
16	16	2	17	4	17	11	19	3
12	19	3	20	6	21	2	22	9
16	17	6	18	8	19	3	20	9
12	20	5	21	9	22	6	24	3
16	18	7	19	10	20	6	22	1

* E is the modulus of elasticity and is in pounds per square inch.

ble XVII. Maximum Span for Floor-Joists for Retail Stores

See explanatory notes on page 736

Distance on centers in	White pine, $S = 1\ 080$ lb per sq in * $A = 60$ ft	in	Norway pine or spruce, $S = 1\ 260$ lb per sq in $A = 70$ ft	in	Douglas fir or Texas pine, $S = 1\ 620$ lb per sq in $A = 90$ ft	in	Long-leaf yellow pine, $S = 1\ 800$ lb per sq in $A = 100$ ft	in
Total load, 174 pounds per square foot								
12	11	6	12	5	14	1	14	9
16	9	11	10	2	12	2	12	9
12	11	8	12	8	14	5	15	1
16	10	2	10	11	12	5	13	1
12	14	4	15	6	17	7	18	7
16	12	5	13	5	15	2	16	0
12	14	1	15	2	17	2	18	2
16	12	2	13	1	14	11	15	8
Total load, 177 pounds per square foot								
12	17	2	18	5	20	11	22	1
16	14	10	16	0	18	2	19	1
12	16	3	17	7	19	11	21	1
16	14	2	15	2	17	3	18	2
12	18	2	19	7	22	3	23	6
16	15	9	17	0	19	3	20	4
12	19	11	21	6	24	5	25	8
16	17	3	18	7	21	2	22	3

e tables is the coefficient of strength for beams and is one-eighteenth of the xural fiber-stress, S. For values of A for other woods, *see Table II, page*

Table XVIII.* Maximum Span for Rafters. Shingled Roofs not Plast

See explanatory notes on page 736

Total load, 48 pounds per square foot										
Sizes of joists	Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$	
in	in	ft	in	ft	in	ft	in	ft	in
2×4	16	7	4	7	9	8	4	9	6
2×4	20	6	7	6	10	7	6	8	6
2×6	16	11	1	11	7	12	6	14	2
2×6	20	9	11	10	4	11	2	12	8
3×6	16	13	7	14	2	15	3	17	5
3×6	20	12	2	12	8	13	8	15	7
2×8	16	14	9	15	6	16	8	18	11
2×8	20	13	3	13	10	14	11	16	11
2×8	24	12	1	12	7	13	7	15	6
2×10	16	18	6	19	3	20	10	23	8
2×10	20	16	7	17	3	18	8	21	2
2×10	24	15	1	15	9	17	0	19	3

Table XIX.* Maximum Span for Rafters. Slate Roofs not Plaster or Shingle Roofs Plastered

See explanatory notes on page 736

Total load, 57 pounds per square foot									
Sizes of joists	Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$	
in	in	ft	in	ft	in	ft	in	ft	in
2×4	16	6	9	7	1	7	7	8	8
2×4	20	6	0	6	4	6	9	7	9
2×6	16	10	2	10	7	11	6	13	0
2×6	20	9	1	9	6	10	2	11	7
3×6	16	12	6	13	0	14	1	15	11
3×6	20	11	1	11	8	12	7	14	3
2×8	16	13	7	14	2	15	3	17	4
2×8	20	12	2	12	8	13	8	15	6
2×8	24	11	1	11	7	12	6	14	2
3×8	16	16	7	17	4	18	9	21	3
3×8	20	14	10	15	6	16	9	19	0
3×8	24	13	7	14	2	15	3	17	4
2×10	16	17	0	17	8	19	2	21	7
2×10	20	15	2	15	10	17	1	19	4
2×10	24	13	10	14	6	15	7	17	8

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* Tables XVIII, XIX and XX are intended for climates where a 2-ft snow-fa expected. In the Southern States, where there is very little snow, the spans XVIII will be safe for slate or gravel roofs if the joists are sawed to the full di Variations in "Safe spans" in different tables, for the same kind of wood, dep the assumed safe flexural fiber-stress or modulus of elasticity or both.

† A in the tables is the coefficient of strength for beams and is one-eighteen allowable flexural fiber-stress, S. For values of A for other woods, see Table 628.

e XX.* Maximum Span for Rafters. Slate Roofs Plastered, or Gravel Roofs not Plastered

See explanatory notes on page 736

Total load, 66 pounds per square foot

Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$		Long-leaf yellow pine, $S=1\,800$ lb per sq in $A=100$	
in	ft	in	ft	in	ft	in	ft	in	ft	in
16	9	5	9	10	10	8	12	1	12	9
20	8	6	8	10	9	6	10	9	11	5
16	11	7	12	1	13	1	14	10	15	7
20	10	4	10	10	11	8	13	3	14	0
16	12	7	13	2	14	2	16	2	17	0
20	11	3	11	9	12	9	14	5	15	2
24	10	3	10	9	11	7	13	2	13	10
16	15	5	16	1	17	5	19	9	20	10
20	13	9	14	5	15	3	17	8	18	8
24	12	7	13	2	14	2	16	2	17	0
16	15	9	16	6	17	9	20	2	21	3
20	14	1	14	8	15	11	18	0	19	0
24	12	10	13	5	14	6	16	6	17	5
16	18	10	19	9	21	4	24	2	25	6
20	16	10	17	8	19	1	21	8	22	10
24	15	5	16	1	17	5	19	9	20	10

XVIII, XIX and XX are intended for climates where a 2-ft snow-fall may be
In the Southern States, where there is very little snow, the spans in Table
be safe for slate or gravel roofs if the joists are sawed to the full dimensions.
in "Safe spans" in different tables, for the same kind of wood, depend upon
d safe flexural fiber-stress or modulus of elasticity or both.
e tables is the coefficient of strength for beams and is one-eighteenth of the
exural fiber-stress, S. For values of A for other woods, see Table II, page

Table XXI. Maximum Span for Ceiling-Joists

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E=1$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 20 pounds per square foot											
2×4	12	8	11	9	3	9	6	9	10	[illegible]	[illegible]
2×4	16	8	1	8	5	8	8	8	11	[illegible]	[illegible]
2×6	12	13	5	13	10	14	4	14	9	15	[illegible]
2×6	16	12	2	12	7	13	0	13	5	[illegible]	[illegible]
2×8	12	17	10	18	6	19	1	19	8	[illegible]	[illegible]
2×8	16	16	3	16	10	17	4	17	10	[illegible]	[illegible]
2×8	20	15	1	15	7	16	1	16	7	[illegible]	[illegible]
Total load, 24 pounds per square foot											
2×10	12	21	0	21	9	22	5	23	1	[illegible]	[illegible]
2×10	16	19	1	19	8	20	5	21	0	[illegible]	[illegible]
2×10	20	17	8	18	4	18	11	19	6	[illegible]	[illegible]
2×12	12	25	2	26	0	26	11	27	9	[illegible]	[illegible]
2×12	16	22	11	23	9	24	6	25	2	[illegible]	[illegible]
2×12	20	21	3	22	0	22	9	23	5	[illegible]	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XXII. Maximum Span for Floor-Joists for Dwellings, Tenements and Grammar-School Rooms with Fixed Desks

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E=1$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 60 pounds per square foot											
2× 6	12	9	3	9	7	9	11	10	3	11	[illegible]
2× 6	16	8	5	8	9	9	0	9	3	10	[illegible]
3× 6	12	10	8	11	0	11	4	11	8	12	[illegible]
3× 6	16	9	8	10	0	10	4	10	8	11	[illegible]
2× 8	12	12	4	12	10	12	3	13	8	14	[illegible]
2× 8	16	11	3	11	8	12	0	12	4	13	[illegible]
3× 8	12	14	2	14	8	15	2	15	7	16	[illegible]
3× 8	16	12	11	13	4	13	9	14	2	15	
2×10	12	15	6	16	0	16	7	17	0	18	
2×10	16	14	1	14	7	15	0	15	6	16	
Total load, 66 pounds per square foot											
3 ×10	12	17	2	17	9	18	4	18	11	20	
3 ×10	16	15	7	16	2	16	8	17	2	18	
2 ×12	12	18	0	18	8	19	3	19	8	21	
2 ×12	16	16	4	16	11	17	8	18	0	19	
3 ×12	12	20	7	21	4	22	0	22	8	24	
3 ×12	16	18	8	19	4	20	0	20	7	23	
2 ×14	12	21	0	21	11	22	5	23	1	24	
2 ×14	16	19	1	19	9	20	5	21	0	22	
2½×14	12	22	7	23	5	24	2	24	11	26	
2½×14	16	20	6	21	3	21	11	22	7	24	
3 ×14	12	24	0	24	10	25	8	26	5	28	
3 ×14	16	21	10	22	7	23	4	24	0	25	

* E is the modulus of elasticity and is in pounds per square inch.

le XXIII. Maximum Span for Floor-Joists for Office-Buildings

See explanatory notes on page 736

Total load, 93 pounds per square foot

Distance on centers	Hemlock, * $E=900\,000$		White pine, $E=1\,000\,000$		Norway pine, $E=1\,100\,000$		Short-leaf yellow pine, spruce, $E=1\,200\,000$		Long-leaf yellow pine, Douglas fir, $E=1\,500\,000$	
in	ft	in	ft	in	ft	in	ft	in	ft	in
12	12	3	12	8	13	1	13	6	14	6
16	11	1	11	6	11	11	12	3	13	2
12	13	4	13	10	14	3	14	8	15	10
16	12	2	12	7	13	0	13	4	14	5
12	15	4	15	10	16	4	16	10	18	2
16	13	11	14	5	14	10	15	4	16	7
12	16	0	16	7	17	2	17	8	19	0
16	14	7	15	1	15	7	16	0	17	3
Total load, 96 pounds per square foot										
12	18	2	18	10	19	5	20	0	21	6
16	16	6	17	1	17	8	18	2	19	7
12	18	6	19	2	19	10	20	5	21	11
16	16	10	17	5	18	0	18	6	19	11
12	19	11	20	8	21	4	22	0	23	8
16	18	2	18	9	19	5	19	11	21	6
12	21	2	21	11	22	8	23	4	25	2
16	19	3	19	11	20	7	21	2	22	10

* E is the modulus of elasticity and is in pounds per square inch.

XIV. Maximum Span for Floor-Joists for Churches and Theaters with Fixed Seats

See explanatory notes on page 736

Total load, 102 pounds per square foot

Distance on centers	Hemlock, * $E=900\,000$		White pine, $E=1\,000\,000$		Norway pine, $E=1\,100\,000$		Short-leaf yellow pine, spruce, $E=1\,200\,000$		Long-leaf yellow pine, Douglas fir, $E=1\,500\,000$	
in	ft	in	ft	in	ft	in	ft	in	ft	in
12	11	10	12	3	12	8	13	1	14	1
16	10	9	11	2	11	6	11	11	12	9
12	12	11	13	5	13	10	14	3	15	4
16	11	9	12	2	12	7	13	0	13	11
12	14	10	15	4	15	10	16	4	17	7
16	13	6	13	11	14	5	14	10	16	0
12	15	7	16	1	16	8	17	1	18	5
16	14	2	14	8	15	1	15	7	16	9
Total load, 105 pounds per square foot										
12	17	8	18	3	18	10	19	5	20	11
16	16	0	16	7	17	1	17	8	19	0
12	18	0	18	7	19	3	19	8	21	4
16	16	4	16	11	17	5	18	0	19	4
12	19	4	20	1	20	8	21	4	23	0
16	17	7	18	3	18	10	19	4	20	11
12	20	7	21	4	22	0	22	8	24	5
16	18	8	19	4	20	0	20	7	22	2

E is the modulus of elasticity and is in pounds per square inch.

Table XXV. Maximum Span for Floor-Joists for Assembly-Halls and

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-l yellow p Dougla $E=1$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 123 pounds per square foot											
3× 8	12	11	2	11	7	11	11	12	3	13	
3× 8	16	11	0	10	6	10	10	11	2	11	
2×10	12	12	2	12	7	13	0	13	5	14	
2×10	16	11	1	11	5	11	10	12	2	13	
3×10	12	13	11	14	5	14	11	15	4	16	
3×10	16	12	8	13	1	13	7	13	11	15	
2×12	12	14	7	15	2	15	8	16	1	17	
2×12	16	13	3	13	9	14	2	14	8	15	
Total load, 126 pounds per square foot											
3 ×12	12	16	7	17	2	17	9	18	3	19	
3 ×12	16	15	1	15	7	16	1	16	7	17	
2 ×14	12	16	11	17	6	18	1	18	7	20	
2 ×14	16	15	4	15	11	16	5	16	11	18	
2½×14	12	18	2	18	10	19	6	20	1	21	
2½×14	16	16	8	17	3	17	10	18	4	19	
3 ×14	12	19	4	20	1	20	8	21	4	21	11
3 ×14	16	17	7	18	3	18	10	19	4	20	

* E is the modulus of elasticity and is in pounds per square inch.

Table XXVI. Maximum Span for Floor-Joists for Retail Stores

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in * $A=33\frac{1}{3}$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, Douglas fir, $S=800$ lb per sq in $A=44.44$		Southern short-leaf yellow pine, $S=1\ 000$ lb per sq in $A=55.55$		Southe long-le yellow p $S=1$ per sq i $A=$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 174 pounds per square foot											
3× 8	12	8	7	9	3	9	11	11	1	12	
3× 8	16	7	5	8	0	8	7	9	7	10	
2×10	12	8	9	9	5	10	1	11	4	12	
2×10	16	7	7	8	2	8	9	9	10	10	
3×10	12	10	9	11	7	12	5	13	10	15	
3×10	16	9	3	10	0	10	9	12	0	13	
2×12	12	10	6	11	4	12	2	13	7	14	11
2×12	16	9	1	9	10	10	6	11	9	12	10
Total load, 177 pounds per square foot											
3 ×12	12	12	6	13	6	14	6	16	2	17	
3 ×12	16	10	10	11	9	12	6	14	0	15	
2 ×14	12	12	2	13	1	14	0	15	8	17	
2 ×14	16	10	6	11	4	12	2	13	7	14	11
2½×14	12	13	7	14	8	15	8	17	6	19	
2½×14	16	11	9	12	8	13	7	15	2	16	
3 ×14	12	16	8	18	0	19	3	21	6	23	
3 ×14	16	14	5	15	7	16	8	18	8	20	

* A in the tables is the coefficient of strength for beams and is one-eighteenth of allowable flexural fiber-stress S. For values of A for other woods, see Table II, p 628.

VII.* Maximum Span for Rafters. Shingled Roofs, not Plastered

See explanatory notes on page 736

Total load, 48 pounds per square foot										
Distance on centers	Hemlock, S=600 lb per sq in † A=33⅓		White pine, spruce, S=700 lb per sq in A=38.88		Norway pine, Douglas fir, S=800 lb per sq in A=44.44		Southern short-leaf yellow pine, S=1 000 lb per sq in A=55.55		Southern long-leaf yellow pine, S=1 200 lb per sq in A=66⅔	
in	ft	in	ft	in	ft	in	ft	in	ft	in
16	5	9	6	3	6	8	7	5	8	2
20	5	2	5	7	5	11	6	8	7	4
16	8	8	9	4	10	0	11	2	12	3
20	7	9	8	4	8	11	10	0	10	11
16	10	7	11	5	12	3	13	8	15	0
20	9	6	10	3	10	11	12	3	13	5
16	11	6	12	6	13	4	14	11	16	4
20	10	4	11	2	11	11	13	4	14	7
24	9	5	10	2	10	11	12	2	13	4
16	14	5	15	7	16	8	18	8	20	5
20	12	11	13	11	14	11	16	8	18	3
24	11	9	12	9	13	7	15	2	16	8

XVIII.* Maximum Span for Rafters. Slate Roofs, not Plastered, or Shingled Roofs, Plastered

See explanatory notes on page 736

Total load, 57 pounds per square foot										
Distance on centers	Hemlock, S=600 lb per sq in † A=33⅓		White pine, spruce, S=700 lb per sq in A=38.88		Norway pine, Douglas fir, S=800 lb per sq in A=44.44		Southern short-leaf yellow pine, S=1 000 lb per sq in A=55.55		Southern long-leaf yellow pine, S=1 200 lb per sq in A=66⅔	
in	ft	in	ft	in	ft	in	ft	in	ft	in
16	5	3	5	9	6	1	6	10	7	6
20	4	9	5	1	5	6	6	1	6	8
16	7	11	8	7	9	2	10	3	11	3
20	7	1	7	8	8	2	9	2	10	1
16	9	9	10	6	11	3	12	7	13	9
20	8	8	9	5	10	1	11	3	12	4
16	10	7	11	5	12	3	13	8	15	0
20	9	6	10	3	10	11	12	3	13	5
24	8	8	9	4	10	0	11	2	12	3
16	13	0	14	0	15	0	16	9	18	4
20	11	7	12	6	13	5	15	0	16	4
24	10	7	11	5	12	3	13	8	15	0
16	13	3	14	4	15	3	17	1	18	9
20	11	10	12	9	13	8	15	3	16	9
24	10	10	11	8	12	6	13	11	15	3

XXVII, XXVIII and XXIX are intended for climates where a 2-ft snow-fall
ected. In the Southern States, where there is very little snow, the spans in
II will be safe for slate or gravel roofs if the joists are sawed to the full dimen-
iations in "Safe spans" in different tables, for the same kind of wood, depend
sumed safe flexural fiber-stress or modulus of elasticity or both.

t-note with Table XXVI.

Table XXIX.* Maximum Span for Rafters. Slate Roofs, Plastered Gravel Roofs, not Plastered

See explanatory notes on page 736

Total load, 66 pounds per square foot										
Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in † $A=33\frac{1}{3}$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, Douglas fir, $S=800$ lb per sq in $A=44.44$		Southern short-leaf yellow pine, $S=1\,000$ lb per sq in $A=55.55$		
in	in	ft	in	ft	in	ft	in	ft	in	
2× 6	16	7	5	8	0	8	6	9	6	
2× 6	20	6	7	7	2	7	7	8	6	
3× 6	16	9	0	9	9	10	5	11	8	
3× 6	20	8	1	8	9	9	4	10	5	
2× 8	16	9	10	10	8	11	4	12	8	
2× 8	20	8	10	9	6	10	2	11	4	
2× 8	24	8	0	8	8	9	3	10	5	
3× 8	16	12	1	13	0	13	11	15	7	
3× 8	20	10	9	11	8	12	5	13	11	
3× 8	24	9	10	10	8	11	4	12	8	
2×10	16	12	4	13	3	14	2	15	11	
2×10	20	11	0	11	11	12	9	14	7	
2×10	24	10	1	10	10	11	10	13	0	
2×12	16	14	9	15	11	17	1	19	1	
2×12	20	13	2	14	3	15	3	17	1	
2×12	24	12	1	13	0	13	11	15	7	

* Tables XXVII, XXVIII and XXIX are intended for climates where a 2-ft s may be expected. In the Southern States, where there is very little snow, the Table XXVII will be safe for slate or gravel roofs if the joists are sawed to the full sions. Variations in "Safe spans" in different tables, for the same kind of wood, upon the assumed safe flexural fiber-stress or modulus of elasticity or both.

† See foot-note with Table XXVI.

To Determine the Strength of an Existing Floor. When a is leased for mercantile or manufacturing purposes the tenant will desire to know the greatest load which it will be safe to put upon th and some building laws require that the safe load for the floors in certai of buildings shall be computed and posted in a conspicuous place in ea It is therefore important that every architect should know how to com safe strength of any existing floor. The problem is practically the r that of proportioning a floor to a given load. In speaking of the stre floor a distinction should be made between the safe strength and the The SAFE STRENGTH should mean the maximum safe load for the beam ing the weight of the construction, flooring and ceiling, while the s refers to the maximum load which may safely be placed upon the flo safe load is found by first computing the safe strength and then su the weight of the materials forming the floor, including the ceiling below is one. The most convenient measurement for either the SAFE STR the SAFE LOAD of a floor is in pounds per square foot. The following will serve to show the method of determining the safe load for an ordin house-floor.

Example 4. It is required to determine the safe load per square floor framed as shown in Fig. 4, the building being in a city the

sq in for the safe flexure fiber-stress for the wood of which
lers are made. The joists are covered with two thicknesses
nd the ceiling below is corrugated iron.

irst step will be to find the SAFE STRENGTH of the 22-ft joists.
house-floor we will use the tables for strength throughout.
, page 643, for $S = 1\,200$ lb per sq in, we find the safe
14-in joist of 22-ft span to be 1 188 lb; hence the strength
n joist will be $1\,188 \times 2\tfrac{1}{2} = 2\,970$ lb. As the joists are
each joist supports a floor-area of $1\tfrac{1}{3} \times 22$ ft $= 29\tfrac{1}{3}$ sq ft.
H PER SQUARE FOOT of this portion of the floor will therefore

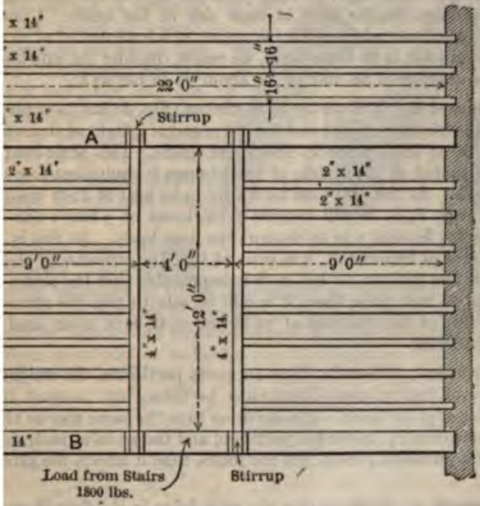


Fig. 4. Plan of a Warehouse-floor

1 lb. Suppose the estimated weight of the floor per square
joists, 6 lb for the flooring and 1 lb for the corrugated-iron
lb in all. Then the SAFE LOAD PER SQUARE FOOT for the 22-ft
$- 15 = 86$ lb.

We will next find the safe load for the 4 by 14-in headers at
stair-well. As the tail-beams are framed into the headers,
one inch from the thickness of each header for the loss of
g, leaving 3 by 14 in for the effective dimension of each.
page 643, we find the safe strength of a 1 by 14-in beam
1 867 lb. Hence the strength of the 3 by 14 will be $1\,867 \times 3$
floor-area supported by each header is $4\tfrac{1}{2} \times 12$ ft $= 54$ sq ft;
RENGTH of the header per square foot of floor is $5\,601/54 =$
g the weight of the floor per square foot, we have $104 - 15 =$
LOAD.

immer A (Fig. 4) supports about the same amount of floor-
ommon joists, and supports, also, the ends of the headers.
thickness of the common joists, we have a 5 by 14-in beam

the 5 by 14-in header is 1 848/27 = 68 lb, and deducting
the weight of the floor, we have 53 lb per sq ft as the safe loa
will support on the floor at each side of the stairs. Con
above, that the safe load for the 2½ in, which we deducted to
common joist, is 86 lb per sq ft, we might consider the safe lo
as the average of 86 and 53, or about 70 lb per sq ft.

Trimmer B. This 10 by 14-in timber (Fig. 4) has to supp
loads as trimmer *A*, and also the lower end of a flight of stairs
ance of at least 1 800 lb should be made. This stair-load
concentrated at the middle of the trimmer is equivalent to a
3 600 lb. As the safe load for a 1 by 14-in joist of 22-ft span
XII, page 643), it will require a thickness of 3 600/1 188
the stairs, leaving 7 in to support the floor-loads. As this is
thickness of trimmer *A*, it is evident that the strength of the
a little less than at *A*; but as it is improbable that the entire
loaded at any given time, it would be safe to rate the stren
each side of the stairway at 70 lb per sq ft, LIVE LOAD, and
way at 86 lb.

Partitions. When the floor supports partitions, the weight
any load resting upon them must be taken into account i
safe load for the floor. If a partition runs the same way as th
the joist directly under the partition, and the joists at each sid
but if a partition runs across the joists, then it affects the safe
floor.

Example 5. Suppose that the 22-ft joists in the floor show
support a plastered partition 12 ft high, running across the j
tween the walls. What will be the safe load for the floor?

Solution. A plastered partition with 2 by 4 or 2 by 6-in s
centers, weighs about 20 lb per sq ft of partition-face; henc
high will weigh 240 lb per lin ft of partition. As the joists
ters, each joist supports 1⅕ lin ft of partition, weighing 320 l
concentrated at the middle span of the joists it is equivalen

ɔards set at right-angles to the joists and fitting in between them. of this bracing is of decided advantage in sustaining any CONCEN- D upon a floor; but it does not materially strengthen a floor to resist LY DISTRIBUTED LOAD. The o stiffens the joists, and pre- from turning sidewise. It is :o insert rows of cross-bridging ft apart; and to be effective bridging should be in straight he floor, so that each bridging- abut directly opposite those it. The method of bridging 'ig. 5, and known as CROSS- s considered to be by far the t allows the thrust to act the axis of the strut, and not grain, as must be the case e pieces of boards are used. ng should be of 1¼ by 3-in by 10-in and smaller joists, 3-in stock for 12- and 14-in

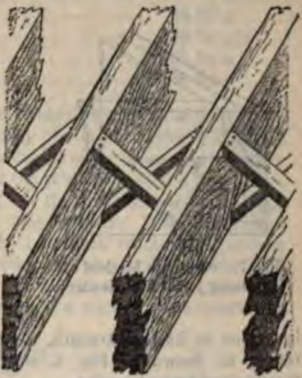
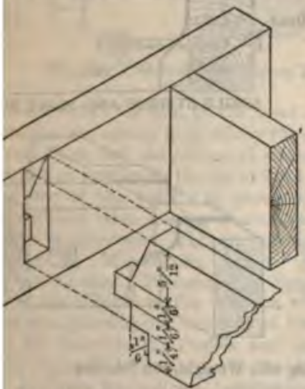


Fig. 5. Floor-joists with Bridging

of Wooden Floor-Beams. ɛs, tenements and lodging- frequently necessary to frame the timbers so that they are flush other. The old methods of framing the tail-beams and headers or trimmers by mortise-and-tenon joints are now generally superseded the timbers in stirrups or malleable-iron joist-hangers. In this construction the entire strength of the timbers is retained, while the cost of the hangers is often less than the labor-cost in preparing the mortise-and-tenon joints. All headers 6 ft or more in length should be carried in joist-hangers or stirrups and this is usually required in the building codes of the large cities. In warehouses and all first-class buildings the framing should be done by means of joist-hangers. For light floors, with moderate spans, it is generally safe to frame the tail-beams into a header, provided the latter is strong enough to carry the load and allow 1 in in thickness for the mortising. Headers, g not more than two tail-beams are often framed into the trimmers. ɔld methods of framing are used instead of the superior methods ngers, the best shape and proportions for the tenons and ends of ns or headers are those shown in Fig. 6. This form of framing



Framing of Joists into Header

probably offers as large a proportion of the strength of the timber possible to utilize, although for tail-beams it was the opinion of Mr. that a single tenon like that shown in Fig. 7 is fully as strong, especially the header is built up of 2-in planks spiked together. In either case, if the

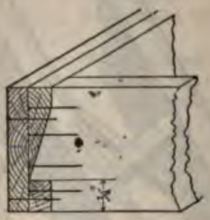


Fig. 7. Alternate Method of Framing Joists into Header

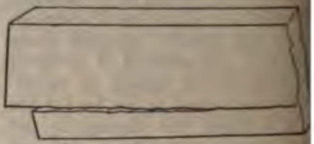


Fig. 8. Framed Joist Split by Load

is loaded to its full strength, the tail-beam will split at the bottom tenon, as shown in Fig. 8, which illustrates the weakening effect mortise-and-tenon framing.

Stirrups and Joist-Hangers. The first device used for framing trimmers without mortising was the wrought-iron stirrup shown in These are made either single or double, depending upon whether one beams are to be supported. To prevent the floor from spreading and thus

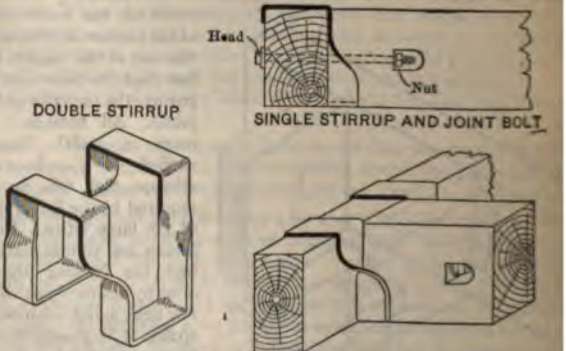


Fig. 9. Framing with Wrought-iron Stirrups

mitting the header to slip out of the stirrup, a joint-bolt may be shown in the two right-hand illustrations of Fig. 9. To determine the of a stirrup, multiply the sectional area of the iron, in square inches, by lb per sq in. (Table 1, page 376.)

The following sizes of iron should, in general, be used for the of joists to be supported:

mbers to be inches	Sections of stirrup-iron in inches
by 10	¼ by 2½
by 12	⅜ by 2½
by 14	⅜ by 3
by 14	½ by 3½
	½ by 4
10 by 14	⅝ by 4

rs. Aside from the matter of strength there are objections to ps. If the timber on which they rest is not perfectly dry, the le by an amount equal to the shrinkage of the beam on which they vn the header with them, and the projection of the iron above imbers will necessitate cutting out the flooring. If the stirrups this way their appearance is objectionable. While they may resist any tensional stress the resistance of steel to bending y small, and the resulting crushing of the timber where they ge is the chief objection to the use of stirrups of this type ded floors. The small bearing of a timber on a stirrup is

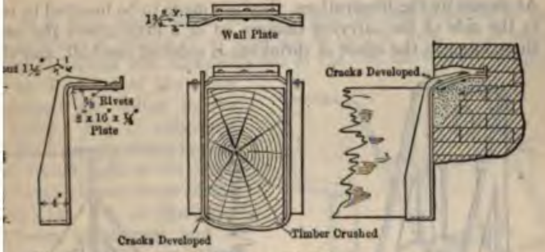


Fig. 10. Failure of Steel Stirrup Wall-hanger

distribute the load on the wood over the required area. This aring per square inch, allows the hanger to crush into the edge raighten out the stirrup as shown in Fig. 29, page 757. The jection applies to the use of steel stirrup-hangers in brick walls free of the walls. As previously explained, all the load is extreme edge, causing a much greater load per square inch on an is allowable. Fig. 10 * shows the effect of crushing, in a ing in Minneapolis, Minn. Wall-hangers made of steel not be used. Patented steel hangers riveted to bearing-plates y undesirable as the crushing effect is greatest at the outer straightening-out tendency of the hanger at this point.

12 illustrate the Duplex and Goetz joist-bangers, which are e claimed to be superior to the old-style stirrups. The Duplex not only for ordinary building-construction, but for the most nill-construction in factories and warehouses. As these hangers lleable iron they will not straighten out when heated, in case of the beams. That is what happens to wrought-steel *stirrups*

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paper on "Joist and Wall-Hangers," read by Mr. F. E. Kidder at a orado Chapter of the American Institute of Architects, February 27,

when the twist becomes heated. This hanger has proven perfectly
and is extensively used. Both are made in sizes to fit all regular
or girders, and have ample strength for the purpose for which they are

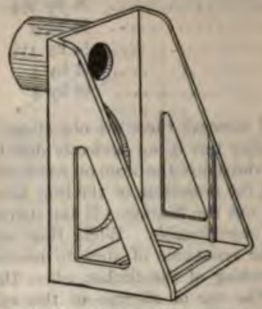


Fig. 11. Duplex Joist-hanger

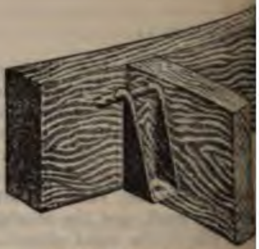


Fig. 12. Goetz Joist-hanger

As shown by the illustrations, they are made to be inserted in round
in the side of the carrying timbers, at or a little above the center
these hangers the effect of shrinkage is reduced one-half, and the other

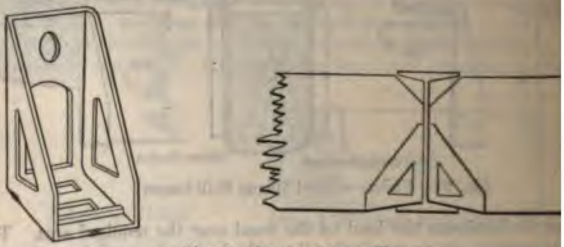


Fig. 13. Duplex I-beam Hangers

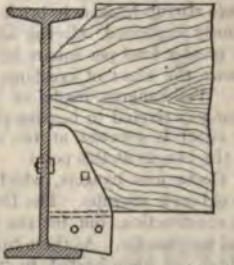
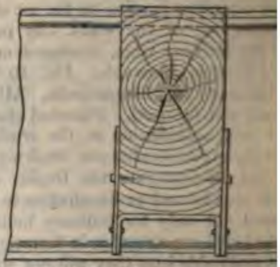


Fig. 14. Duplex I-beam Shelf-hanger.

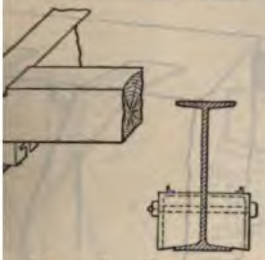


Joists Raised Less than Four

jections to the stirrup, previously mentioned, are overcome.
hanger has ridges on the inside of the side brackets to hold the

For timbers of larger size and for the heaviest construction, the

32, page 789, are used and are bolted to the beams. By this
e entire building is tied together laterally.
s the Duplex I-beam hanger for framing floor-joists to I beams.
made to exactly fit into the flange of the I beam. It has a rib



x I-beam Box Hanger. Joists
More than Four Inches

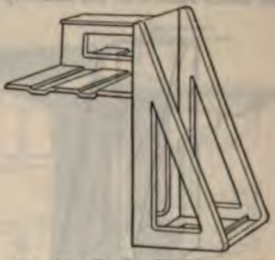


Fig. 16. Duplex Wall-hanger for Joists

¾ in high, which serves as a tie when the joist is placed in the
provides a bearing of at least 4½ in for the joist. It is made
ist of regular size, and offers one of the best devices for framing
to I beams of the same
angers are bolted to the
beam. Fig. 14 shows the
shelf-hanger which is used
ruction requires the joists
ve the lower flange of the
n 4 in. Fig. 15 illustrates
beam box-hanger and is
vhere the joists are raised
n above the lower flange
In both these construc-
ers are bolted singly or
uired, on the I beam and
carried on the lower flanges of the beams. Fig. 16 shows a
made to support the wall-end of a floor-joist. This form of
considered much superior to the method of building the

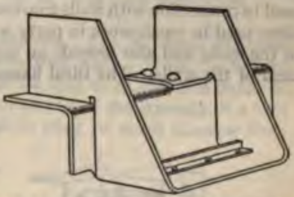
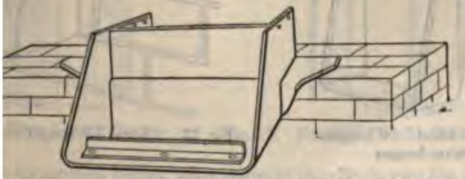


Fig. 17. Duplex Steel Wall-hanger for Large Beams



l8. Duplex Extra-heavy Wall-hanger for Mill-construction

all, as it absolutely *prevents dry-rot, and permits the joists to*
re, without throwing the wall. It also gives the load a good
ll. Fig. 17 illustrates the Duplex steel wall-hanger for larger
8 shows the Duplex extra-heavy wall-hanger for the heaviest

mill-construction. These hangers bear the label of approval of th
Board of Fire Underwriters and are generally considered the best-des
hangers now on the market. This hanger gives an extra bearing on th
and is so constructed that it reacts as a unit and distributes the load e
the entire surface of the masonry. There is no tendency for a hang
type to crush in at the edge of the masonry and straighten out, as
with some other types of wall-hangers. Fig. 19 shows the Duplex
used in connection with walls constructed of concrete blocks. These
often used in repair-work in party walls, as they avoid the cutting of
in the walls, and also provide an easy and simple method of carryin
clear of the walls. The Ideal hanger illustrated in Fig. 20 is made
steel and corrugated at the points where it is bent over. This reinf
tends to prevent bending at these points. Fig. 21 illustrates anot
the Ideal hanger with holes for spiking to a timber. This hang
corrugated. In these hangers the full strength of the steel is retai
fibers of the metal are not cut in forming them. They are made
steel bars folded to the required shape. Fig. 22 shows the Ideal

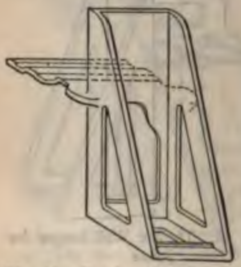


Fig. 19. Duplex Wall-hanger for Concrete Blocks

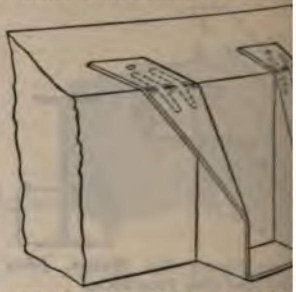


Fig. 20. "Ideal" Wrought-steel B

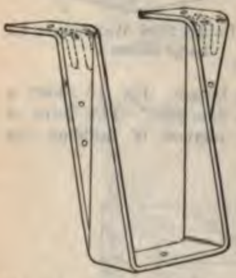


Fig. 21. "Ideal" Wrought-steel Beam-hanger

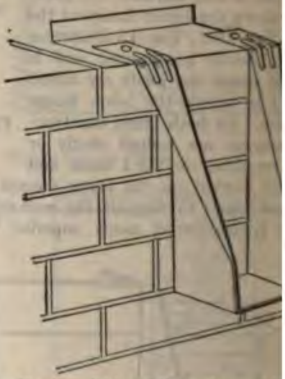
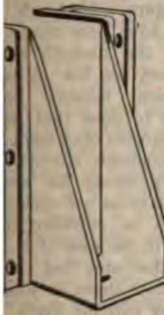


Fig. 22. "Ideal" Wrought-steel W

late and in position to be built into a brick wall. Other illustrations
gers are given in Chapter XXII. The Van Dorn hanger, illustrated
is essentially a stirrup forged from high-grade steel. The few tests
been made would seem to indicate that it developes a greater re-
bending than the ordinary stirrup, while it gives a wider bearing



an Dorn Beam-hanger

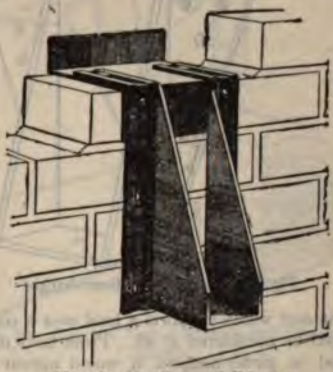
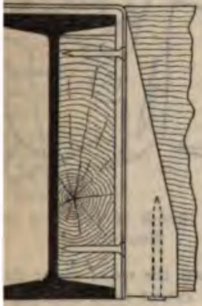


Fig. 24. Van Dorn Wall-hanger

ist and presents a much neater appearance. Fig. 24 shows the same
eted to a bent iron plate, to build into brick walls. When the hanger
sed over a steel beam the upper ends are bent to fit over the flange of
, as in Fig. 25. "Although I know of no test of the strength of a Van
eam hanger, it would seem as though it must be much stronger than



5. Van Dorn I-beam Hanger

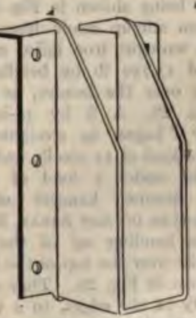


Fig. 26. National Joist or Beam-hanger

rn made for wooden beams, on account of the clinch over the flange of
m. The Van Dorn hangers have been used in many important build-

6 and 27 show the *general form of two other patented joist-hangers,*
forged from plate steel. Both of these hangers, also, are made to be

---

*F. E. Kidder.

built into brick walls and to go over steel beams. The National hange
has a flange on top, which helps materially in distributing the load o
of the beam as shown in figure. The larger hangers of this style ha

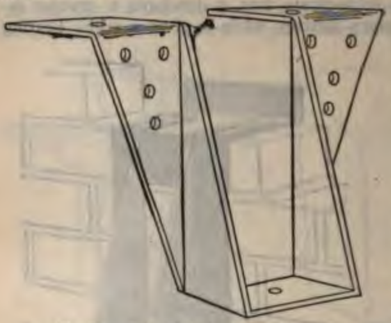


Fig. 27. Lane Joist or Beam-hanger

the top for large sp
hanger and the La
(Fig. 27) have been

**Comparative Str
Different Types
Hangers.** Although
that have been mad
mine the strength o
hangers are few in
sufficient number
made to show that
the hangers described
the common stirrup
dantly strong for
FLOOR-BEAM not exce
14 in in cross-secti
only in the case of a
trimmer which supports a load over a considerable floor-area that t
need be considered at all. From tests made at various times on jo
and on girder-hangers, it would appear that, under extreme loads
hangers usually develop great strength. A two-part hanger, carr
by 14-in girder, sustained a load of 38 000 lb without injury to
itself. A similar hanger held until
loaded up to 39 550 lb, when one
side broke off short under the
nipple projecting into the timber,
the condition of the hanger after
failure being shown in Fig. 28. A
common stirrup made from ⅜ by
2½-in wrought iron failed under a
load of 13 750 lb by bending and
pulling over the header, as shown
in Fig. 29. A 6 by 12-in steel
hanger "began to straighten out
under a load of 13 300 lb, and failed
to hold under a load of 18 750
lb."* SINGLE hangers of the
stirrup-type DO NOT BREAK, but fail
by the bending up of the parts
which lie over the top of the header
as shown in Fig. 29. They also appear to crush the wood under
ticularly at the edges, to a very much greater extent than does t
the Duplex hanger. With a DOUBLE stirrup the ultimate strength i
by the strength of the iron. Thus, a double stirrup, made of ⅜ by 2½-
iron, was loaded up to 57 650 lb (28 825 lb on each side), when it b
of the lower corners. A single stirrup would of course be just as
could be kept from bending. In actual construction the flooring over
to some extent prevents the top of a stirrup from springing up. Th

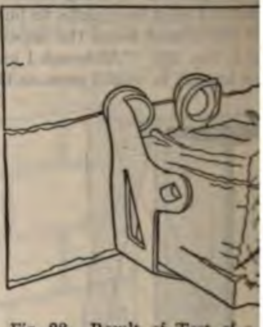


Fig. 28. Result of Test of a
Beam-hanger

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* From data compiled by Mr. Kidder from a series of tests on
and joist-hangers.

nade of two-part hangers show conclusively that where only a
r is used the holes
ored in the header do
y affect its strength
ad is within the safe
test made at Balti-
August 24, 1904, with
oists, spaced 12 in on
suspended by these
into a header formed
y 12-in joists, spiked
ould seem to prove
ien the holes are 12 in
not seriously weaken
nly record of the fail-
orm of hanger when
se in a building, of
aware, is that of a
Iinneapolis, where a
ix floors of a ware-
Nov. 7, 1902, through
a wall-hanger made
2 by ¼-in structural-
which was sheared and bent, and riveted to an 8 by 16 by ¼-in
. The failure was due to the crushing of the outer edge of the
der the hanger, and the consequent bending up of the top portion.
ad on the hanger was about 15 000 lb." *

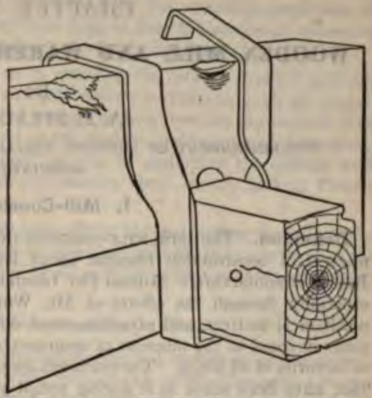


Fig. 29. Result of Test of Wrought-iron Stirrup-hanger

E. Kidder. See, also, Engineering News, Nov. 20, 1902.

1. Mill-Construction

Definition. The term MILL-CONSTRUCTION is commonly u
method of construction brought about largely through th
Boston Manufacturers' Mutual Fire Insurance Company of B
especially through the efforts of Mr. Wm. B. Whiting, w
mechanical matters, and experience and skill as a manufactu
years devoted to the interests of insurance companies, and to
of factories of all kinds. The extended use of this system and
that have been made in it during recent years are probabl
influence of Mr. Edward Atkinson, President of the Bosto
Mutual Insurance Company and Director of the Insurance E
ment Station at Boston, than to that of any other individual.

Cost. The purpose of mill-construction is to reduce the f
est point without going to the expense of fire-proof constructio
cost of heavy timber, however, and in fact of all lumber,
lessened cost of the erection of the so-called FIRE-PROOF T
entirely of reinforced concrete, or built with protected steel f
bustible floors, and the recognition, also, of the obvious ad
FIRE-RESISTING CONSTRUCTION, especially in the congested sec
bringing these types into more general use. The cost of the
construction is, in many instances, no more than the cost of
mill-construction.

The Slow-burning or Mill-Construction Type. The e
has entirely justified the use of this type. It renders possibl
costly, and at the same time, what is of great importance, a mo
of fire-protection than can be installed in buildings of light c
the so-called JOISTED FLOORS and with the roofs made of bo
[illegible] joists. The entire subject of

ops. It consists in separating every floor from every other
nbustible STOPS, by installing automatically closing hatchways
ng stairways either in brick or other incombustible partitions, so
be retarded in passing from floor to floor to the utmost consistent
f wood or any material not absolutely fire-proof.

etardants. It consists in guarding the ceilings over all specially
ck or processes with FIRE-RETARDANT MATERIALS, such as plaster-
wire lath or expanded metal, or over wooden dovetailed lath,
lines of the ceilings and of the timbers and leaving no interspaces
plastering and the wood; or else in protecting the ceilings over
aces with asbestos, air-cell boards, sheet metal, Sackett Plaster
er fire-retardant.

afeguards. It consists not only in so constructing the mill, work-
house that fire will pass as slowly as possible from one part of the
nother, but also in providing all suitable SAFEGUARDS AGAINST FIRE.

3. What Mill-Construction Is Not

aled Spaces. Mill-construction does not consist in so disposing a
y of materials that the whole interior of a building becomes a SERIES
CELLS, or concealed spaces, connected with each other directly or
rough which fire may freely pass where it cannot be reached by

f Timbers, Fire-Stops, etc. It does not consist of an open-timber
of floors and roofs which resembles mill-construction, but which
light timber of insufficient size and with thin planks, without fire-
guards from floor to floor.

ays. It does not consist in connecting floor with floor by COM-
ODEN STAIRWAYS encased in wood less than two inches thick.

ions. It does not consist in putting in very numerous LIGHT,
ISIONS or partitions.

hing and Furring. It does not consist in SHEATHING brick walls
specially when the wood is set off from the walls by FURRING, and
are stops behind the furring.

sh. It does not consist in permitting the use of VARNISH on wood-
hich a fire will pass rapidly.

, Fire-Shutters and Wire-Glass. It does not consist in leaving
posed to adjacent buildings and unguarded by FIRE-SHUTTERS or

ing and Dry-Rot. It does not consist in painting, varnishing,
casing heavy timbers and thick planks, as they are customarily
nd thus making possible what is called DRY-ROT, caused by a lack
n or opportunity to season.

klers, Pumps, Pipes, Hydrants, etc. It does not consist in
the best-constructed building in which dangerous occupations are
hout AUTOMATIC SPRINKLERS, and without a complete and adequate
f PUMPS, PIPES and HYDRANTS.

shing in Wood and Other Materials. *It does not consist in
WOOD IN FINISHING a building after the floors and roof are laid than
necessary, since there are now many safe methods available at low
ing walls and constructing partitions with slow-burning or in-*

times been called COMBUSTIBLE CONSTRUCTION.

4. Standard Mill-Construction

Example of Standard Mill-Construction. Fig. 1 show
through a mill of the customary or STANDARD TYPE recommend
Manufacturers' Mutual Insurance Company, the details of c
revised to May, 1908.

Walls. If additional stories are required, the walls ma
thickness according to the number of stories added, after a
been made of the loads which a STANDARD FACTORY may be c
tain. Walls should be of brick and at least 13 in thick in th
their thickness should be increased in the lower stories to s
loads. Plastered walls are often to be preferred to unplastere
arches and door-arches should be of brick, and window-sills, out
under-pinning of granite or concrete.

Roofs and Floors. The roofs should be of 3-in pine pla
to the heavy roof-timbers, and covered with five-ply tar-a
Roofs should incline from ½ to ¾ in per ft, and incombu
recommended when there is exposure from neighboring
should be of spruce planks, 4 in or more in thickness acco
loads, spiked directly to the floor-timbers, and kept at least ½
face of the brick walls. In order to obviate the danger of c
which sometimes results from the swelling of planks laid c
these spaces left between walls and floor-planks must be co
battens both above and below. In floors and roofs, the ba
8 to 10½ ft wide, and all planks two bays in length should
joints every 4 ft, and grooved for hard-wood splines. Usua
birch or maple is laid at right-angles to the planking, but the
double overfloor, a lower one of soft wood, laid diagonally up
an upper one laid lengthwise. This latter method allows b

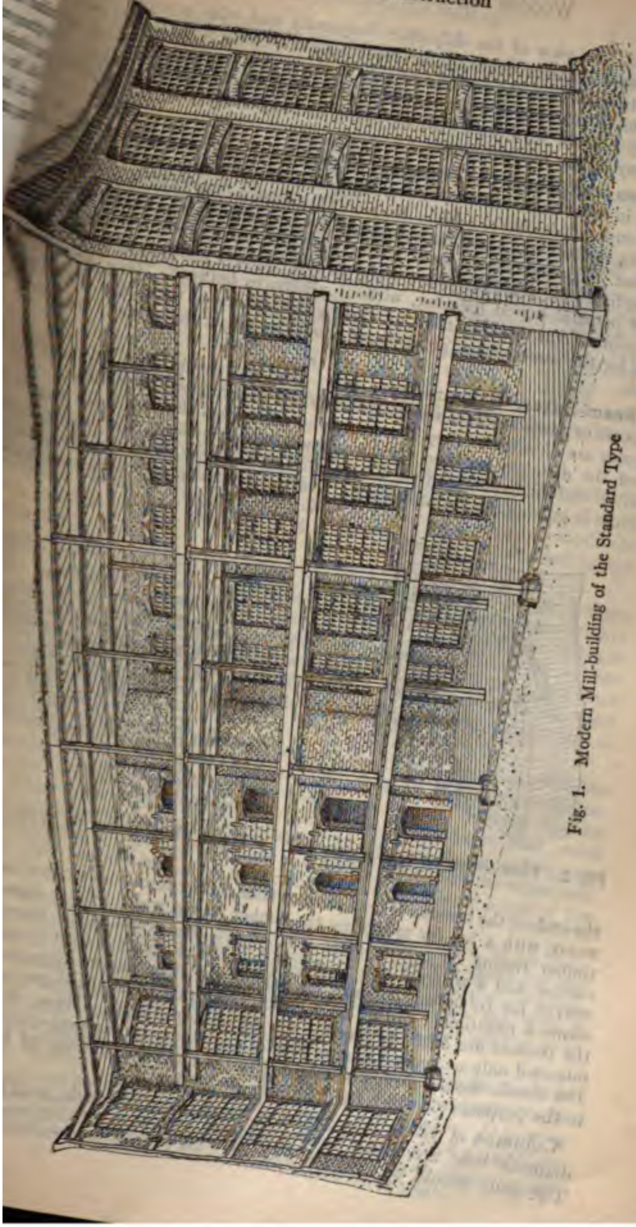


Fig. 1. Modern Mill-building of the Standard Type

provided. In view of the difficulties frequently met with in presen
ment floors of the ordinary timber construction, because of the lack
ventilation underneath, and also in view of the rapid decay of timber
floors in bleacheries, dye-works, print-works, and the like, in which
quickly become saturated with moisture, artificial-stone floors are
in many of the modern plants.

Sizes and Kinds of Timbers. All woodwork, not STANDARD
TION, in order to be SLOW-BURNING, must be in LARGE MASSES which
least surface possible to a fire. No pieces less than 6 in in width sh
for the lightest roofs, and for substantial roofs and floors much wide
needed. Timbers should be of sound, long-leaf, yellow pine, and fo
to 14 by 16 in, single pieces are preferred; or, timbers 7 to 8 by 16 in,
used in pairs bolted together, without air-spaces between. They
be painted, varnished or filled for three years because of the danger
and for the same reason, an air-space should be left in the masonry
ends.

Beam-Boxes, Column-Caps, etc. Timbers should rest on
PLATES or BEAM-BOXES in the walls and on cast-iron caps on the column
BOXES are of value as they strengthen the walls when the floor loads
and the distance between windows small; they facilitate the lay
bricks and the handling of the beams; and there is less danger of b
bricks in putting the beams in place. They also insure proper air-spa

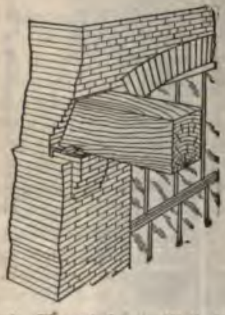


Fig. 2. Floor-timber on Wall-plate

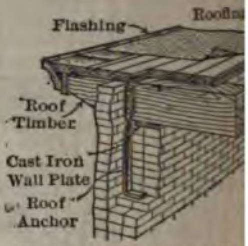
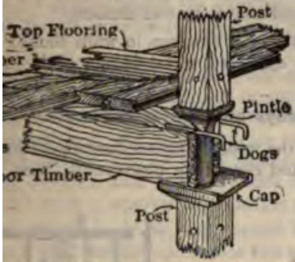


Fig. 3. Roof-timber on Wa

the ends of the beams. Fig. 2 shows a floor-timber resting on a CAST-
PLATE with a lug for anchoring the timber to the wall. Fig. 3 sho
timber resting on a CAST-IRON WALL-PLATE, an overhanging, ope
cornice and a wrought-iron joist-anchor. Fig. 4 shows a CAST-IRO
PINTLE for columns, and dogs for holding the floor-timbers togethe
shows a roof-timber resting on a COLUMN-CAP cast to fit the slope o
the timbers are held together by 1-in wrought-iron dogs. These di
intended only as general illustrations of SLOW-BURNING or MILL-CONS
The details should always be adapted to the special conditions of th
to the purposes for which the buildings are used.

Columns of yellow pine should be bored through the axis, makin
diameter hole, and should have ½-in lateral vent-holes near the top
The ends should be carefully squared. To prevent dry-rot,

e painted until they are thoroughly seasoned. They should be set
vhich may be cast in one piece with the cap, or separately. CAST-
ıs are preferred by some engineers, and when a building is equipped
ıtic sprinklers, such columns have proved satisfactory; but they



t-cap and Pintle for Floor-timber and Columns

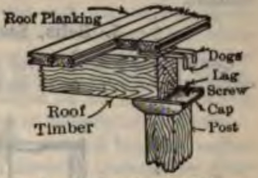


Fig. 5. Roof-timbers on Column-cap

re-resisting as wooden columns. WROUGHT-IRON OF STEEL COLUMNS
e used unless encased with at least 3 in of fireproofing.

ı should be placed as high and made as wide as possible to obtain the
ount of light, and the use of RIBBED GLASS is recommended for the
s.

Deflection and Vibration. In computing the size of the timbers
the working-load, consideration must be given not only to the weights
be carried, but also to the CHARACTER OF THE MACHINERY which is
ed on the floors. Beams of sufficient strength to support the weights
or deflect under the weight and action of the machinery; and there
e, three factors, WEIGHT, DEFLECTION and VIBRATION, which must
ed in determining the width and depth of the beams that are to
he structure.

able Types of Construction. "We do not approve what has
imes miscalled MILL-CONSTRUCTION, that is, longitudinal girders
ı posts and supporting floor-beams spaced 4 ft, more or less, on
is mode of construction not only adds to the quantity of wood
e disposal of the timbers obstructs the action of the sprinklers, pre-
veeping of a hose-stream from one side of the mill to the other, and
also obstruct the most important light, that from the top of the

Ventilation, Painting, etc. Timbers, unless known to be thor-
oned, should not be encased in any kind of air-proof plastering nor
h oil-paints; white-wash, calcimine and water-paints may be used,
porous. As a rule, timbers should be LEFT UNPROTECTED, since a
vill seriously impair and destroy heavy timbers will already have
k upon other parts of the structure.

d Compound Beams. While, in general, SINGLE BEAMS should be
ne instances it may be desirable to substitute *COMPOUND BEAMS*,
stening two or more beams *or thick planks side by side. It is often
tain well-seasoned lumber in small dimensions. Such* COMPOUND
be tightly bolted together without air-spaces, and owing to the
rot, should not be painted or varnished for three years

Steam-Pipes. If a mill is to be heated by conveying steam
such pipes should be hung overhead.

Cornices. Wherever buildings are exposed or are liable to be
fire in the near future, the cornices should be of non-combustible
preferably, the walls should extend above the roof-timbers.

Glass, Frames and Shutters. All openings in walls should be
either by approved wire-glass in approved, metal frames or by
shutters.

5. Belts, Stairways and Elevator-Towers

Continuous Floors. One of the most important features of
CONSTRUCTION is to make each and every floor CONTINUOUS from

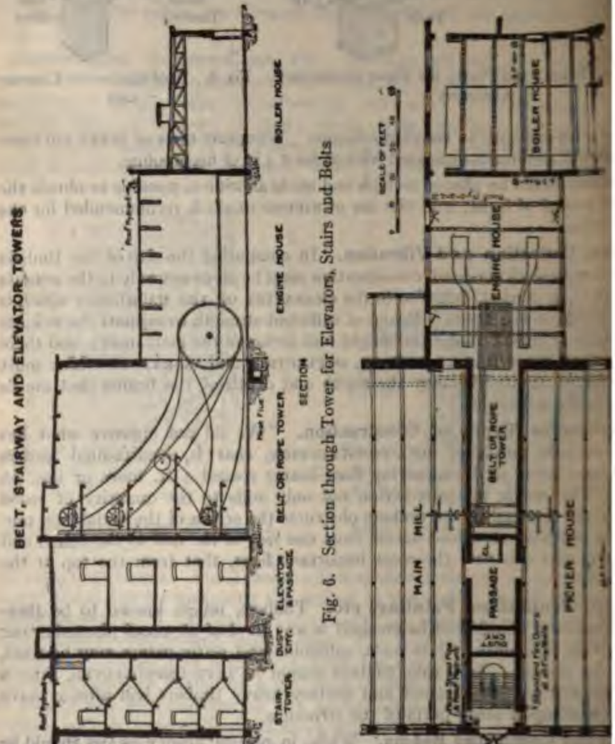


Fig. 6. Section through Tower for Elevators, Stairs and Belts

avoiding, as far as possible, holes for belts, stairways, or elevators
fire may be confined to the story in which it starts. No
owner, engineer or builder will, therefore, fail to locate
main belts, in BRICK TOWERS or in sections of the building cut off

ible walls. All openings in these walls should be protected by E-DOORS, preferably self-closing. In modern practice all belts h may be used for the transmission of power to the various rooms, INCOMBUSTIBLE VERTICAL BELT-CHAMBERS, from which the power by shafts through the walls into the several rooms of the factory. be no unprotected openings in the inner walls of this BELT-

ove Roof. Skylights. All SHAFTS for STAIRS, ELEVATORS, hould extend at least 36 in above the roof, and all such shafts ossible, on the outside of the building. Elevator and belt-shafts ered with thin glass skylights in metal frames, protected underre netting. Figs. 6 and 7 illustrate a section and plan of a COTTON- elevator, stair and belt-shafts arranged on the above principle. ld be in a separate tower rather than in manufacturing rooms.

-Plant should be in a separate building cut off from the enginek wall, and the openings in this wall should be protected by AUTO-G, STANDARD FIRE-DOORS.

6. Standard Storehouse-Construction

f Storehouse-Construction. Fig. 8 shows a cross-section through and Fig. 9 the first-story plan, including the elevator and stair-ur-story uildings purpose in gen-; 000 sq When er, for on-haz-ls, the be in-000 sq

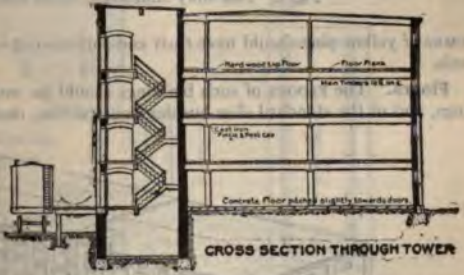

Hard wood top Floor
Floor Plank
Concrete Floor pitched slightly towards doors
CROSS SECTION THROUGH TOWER

Fig. 8. Four-story Storehouse. Section through Fire-tower

Stories. es, the be made low enough (Fig. 10) to prevent overloading, and when ase-goods, the HEIGHT OF STORIES should be sufficient to take two 12-in, clear space under the beams to allow for the distribution of e sprinklers.

. For convenience, as well as to separate the different hazards of and finished goods, the building should be divided into sections s extending at least 36 in above the roof.

Storehouses. A ONE-STORY STOREHOUSE is recommended in the design just described, whenever there is a sufficient quantity at disposal for this purpose. The one-story building is cheaper, ent, and, when separated into small divisions by fire-walls, represt method of storehouse-*construction.*

nd Framing. The FLOOR-TIMBERS *and* ROOF-TIMBERS *should be low pine, in single pieces, if possible. If necessary to use double uld be bolted together without air-spaces between them.* Tim-

bers should rest on cast-iron plates or beam-boxes in the walls, and on ca
caps on the columns. At least ½-in air-spaces should be left around all b
built into the masonry, allowing free ventilation and preventing dry rot.

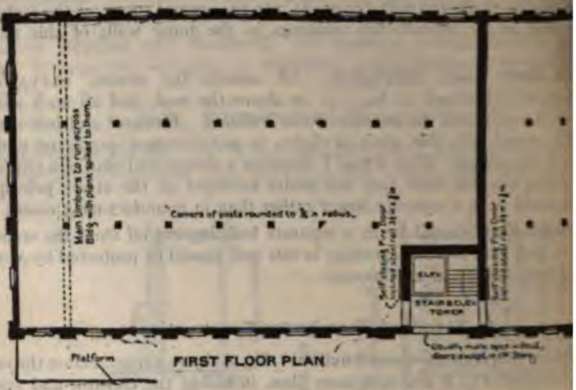

Main timbers to run across
Bldg with plank spiked to them.
Corners of posts rounded to ⅛ in radius.
Self closing Fire Door
ELEV.
STAIR & ELEV. TOWER
Platform
FIRST FLOOR PLAN

Fig. 9. Four-story Storehouse. First-story Plan

UMNS of yellow pine should have their end-surfaces cut square with the c
axis.

Floors. The FLOORS of such buildings should be continuous, withou
ings, and of the standard slow-burning construction, described under St

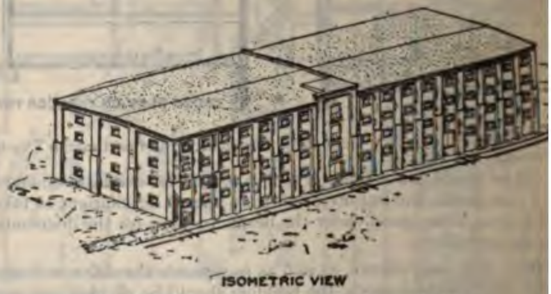


Fig. 10. Four-story Storehouse. Isometric View

MILL-CONSTRUCTION. The flooring should be constructed as called
STANDARD MILL-CONSTRUCTION. In order that the floors may be
water-proof as possible, tarred paper, mopped with tar, should be a
previously suggested. The floors in each story of the tower should b
1 in lower than the floor in the adjoining compartment, and the sills of
openings to the tower should be inclined to make up the difference
The sill, also, of the outside door of the tower should be lower tha
floor.

ater on the floors of the tower will ordinarily flow down the
the arrangement of the floor-levels indicated above will ordi-
ter from an upper story from flowing into one of the lower
it is escaping through the tower. Cast-iron SCUPPERS are
should be set in the brickwork at frequent intervals, and so
will carry away rapidly a maximum quantity of water from
compartment. To further the drainage of water, the floors
from the middle of the compartments to the scuppers. Fig.
-SHIELD SCUPPER* which embodies the latest improvements.

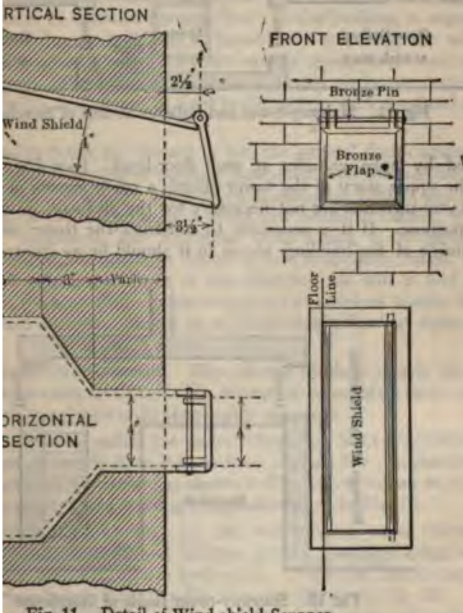


Fig. 11. Detail of Wind-shield Scupper

upper only one flap is provided on the outside of the building.
windy weather, this flap blows open and sometimes freezes
ts in a continuous draft through the scupper and over the
he factory or warehouse and necessitates an increase in the
rnished. The scupper shown in Fig. 11 corrects this condi-
the light wind-shield on the floor-level of the scupper. When
ws open the wind-shield shuts off the draft from the outside.
ddition, acts as a fire-retardant when an adjoining building
en there is a tendency for the flames to communicate through
and ignite merchandise on the floor. *The wind-shield, by
rafts and fire, acts as a retardant or shield to keep out the*

---

the Wind-Shield Scupper Company, 1 Madison Avenue, New

Tower for Stairways, Elevators, etc. Access to the various obtained by means of a BRICK TOWER outside the main building, extending above the roof, and containing STAIRWAYS, ELEVATORS, ETC., access to

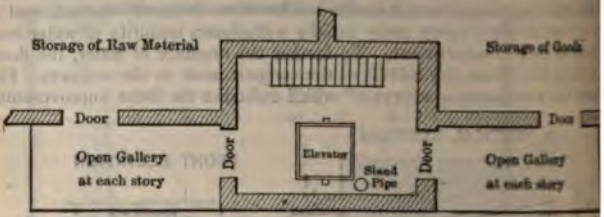


Fig. 12. Stairway-tower and Galleries at Side of Storehouse

obtained by open galleries at each floor-level. (See Fig. 12.) A from the upper story of the tower affords a ready means of reaching AUTOMATIC HATCHES are not necessary for the elevator, as GUARD-GATES every purpose. If it is necessary to construct the tower for the stairs inside of the building, access to it should be as shown in Fig. 13.

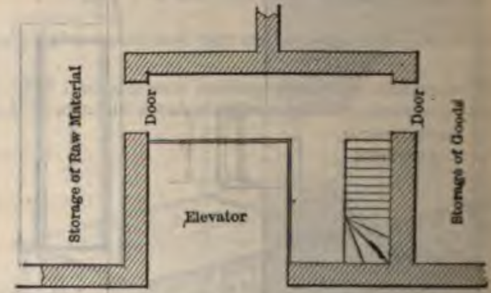


Fig. 13. Stairway-tower Inside of Storehouse

construction serves, also, as a FIRE-TOWER, part of the outside wall omitted.

Roof Walls and Parapets. The WALLS should extend 36 in above and the PARAPET should be laid in cement, because the moisture sorbed by the bricks would otherwise pass downward and make the walls top story damp. In some instances a course of bricks dipped in coal laid above the roof-level.

Sprinklers, Standpipes and Hose. Mills and storehouses should tected throughout by AUTOMATIC SPRINKLERS and by inside HOSE-EQUIPMENTS. Dry-pipe sprinklers should never be used unless it practicable to heat the building. These systems should be planned and vised by a thoroughly reliable fire-protection engineer. (See, also, XXIII, pages 908 to 910.)

7. Example of One-Story Work-Shop

'. For work-shops on cheap, level land, and especially for buildings
e stock is heavy, ONE-STORY BUILDINGS have proved to be more
than higher buildings, in cost of floor-area, supervision, moving
cess of manufacture and repairs to machinery, much of which
greater speeds than when it is in high buildings.

and Ventilating. Window-Area. Such buildings are readily
ventilated, and heavy-plank roofs are free from condensation in
Window-areas should be as large as practicable, as a large window-
he hours of artificial illumination. If the building is exposed to
ner building or buildings of hazardous occupancy, the windows
he Fenestra, Lupton or other equally good, steel construction,
-glass. The forced circulation of heated air is a very desirable
g mills, and should be used in connection with overhead steam-

ooden floors are subject to rot, the general floor-construction,
l be of concrete or earth or some other non-combustible mate-
dust rising from floors of such materials injures machinery,
g of oils weakens such floors and seems to make a WOODEN
necessary, the following construction is recommended.
e, several inches in thickness and thoroughly rolled, is first
r this a 4-in layer of tar-concrete. On this is laid a 1-in
evenly rolled. Over this, 2 or 3-in hemlock planks, bedded
d and over them a ⅞ or 1⅛-in maple floor, at right-angles

n-Construction. Figs. 14 and 15 show clearly the mode
-CONSTRUCTION. No beams or other structural timbers
arnished until thoroughly seasoned.
be as called for under STANDARD MILL-CONSTRUCTION.
dinarily from 8 to 20 ft on centers, the 3-in planks span-
n the trusses as shown in Fig. 14, or resting on PURLINS
ers, and running longitudinally, as in Fig. 15.

rs. In Fig. 14, the overhanging OPEN CORNICE is
e outside and without gutters. Roofs sloping back to
n Fig. 15, are preferable. Projecting BRICK CORNICES,
ork from outside fires, are shown in Fig. 15. If the
r buildings of hazardous construction and occupancy,
and cornices are needed.

he roof-planks should be at least two bays in length,
or, if purlins are used, the planks should cover at
e purlins, and break joints as above. Roof-timbers
walls in a safe and suitable manner. While the
ay be used with this type of building, it may
dvisable; and the types shown in Figs. 14 and
chine-shops, foundries, and similar buildings, in
required for traveling cranes. The middle sec-
ovided with SAW-TOOTH SKYLIGHTS with excellent
thers are made higher for galleries.

In ordinary one-story machine-shops, or in
e wide spans or trusses are necessary, the
s not objectionable.

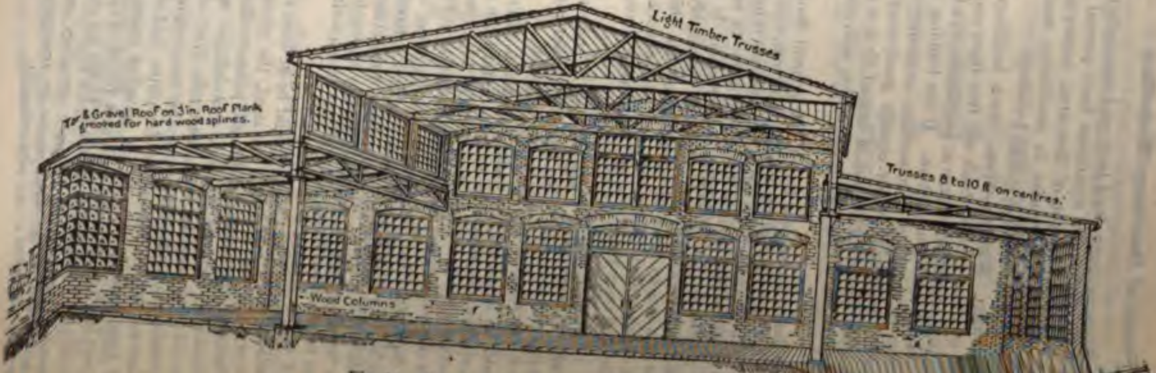
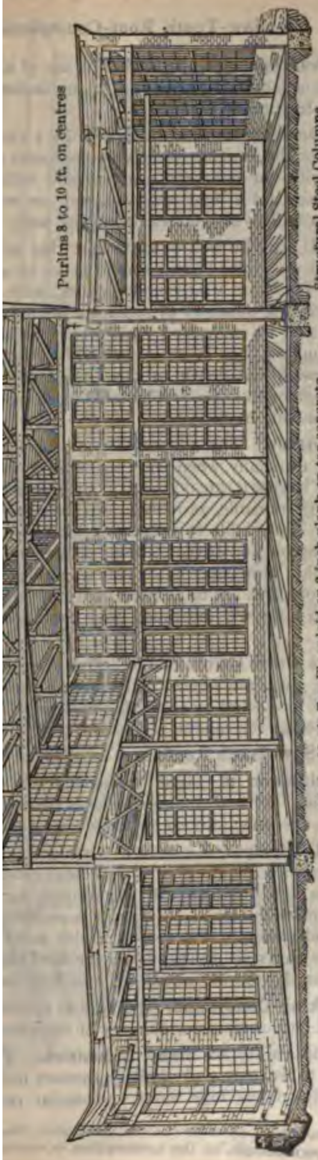

Light Timber Trusses
Tar & Gravel Roof on 3 in. Roof Plank grooved for hard wood splines.
Trusses 8 to 10 ft on centres.
Wood Columns

Fig. 14. One-story Work-shop. Roof-boards on Trusses



1¾ in. Maple Top Floor laid on 2-inch plank on tar concrete, with thoroughly mopped tarred felt under plank

Fig. 15. One-story Work-shop. Roof-boards on Purlins

using light wooden trusses or reinforced-concrete walls, are appl

Roof-Types. It may be well to state here that while light
and 3-in joists and with light boards should never be used, and
ciples of SLOW-BURNING or MILL-CONSTRUCTION, with its hea
preferred, the increasing difficulty of promptly obtaining yellow
good dimensions, and its increasing cost, often necessitate the
and rather light timbers; but in no case should these timbers b
in width nor of insufficient depth to carry the load. This, a
that they may be SLOW-BURNING. The roofs in all cases should
of planks and have wide bays.

Steel Roof-Trusses. The adaptability of the light forms
FRAMING TRUSSES, especially when wide spans are needed, ofte
use; and in plants having a safe occupancy, such as that of meta
trusses are not objectionable, providing adequate sprinkler-pr
good water-supply is available to prevent quick failure of the
to heat from the combustion of the contents of the building or fr
of the roof. Similar protection is, of course, needed in shops
TRUSSES, if disastrous fires are to be prevented; but experience
the STEEL-TRUSSED ROOF will fail much more rapidly than one
similar conditions.

Wooden versus Steel Columns. WOODEN POSTS are nearl
able and should be given preference; but if light STEEL COLUMN
they should be well protected by insulating materials if they ar
taining combustibles, as the column is the vital part of the roof-s

Advantages of Saw-Tooth Roofs may be outlined as follows

(1) **Uniform Diffusion of Light** throughout the room, th
space in it available. With all interior surfaces painted white a
glass in the sashes, the DIFFUSION OF LIGHT is almost perfect.

(2) **Better and Cheaper Lighting.** Greater adaptabilit

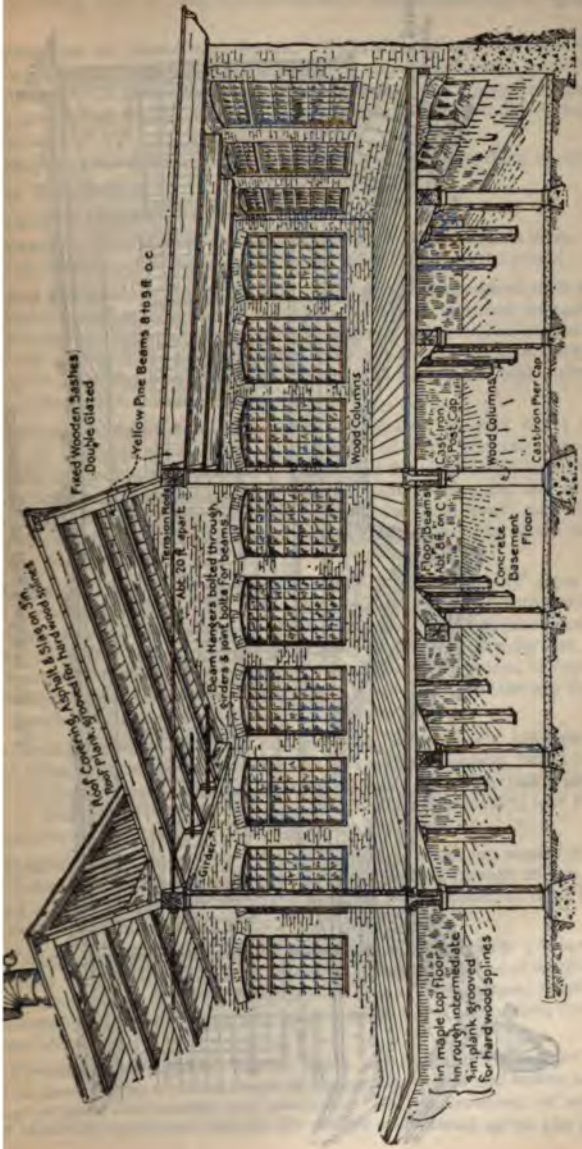

Roof Covering, Asphalt & Slag on 5 in. Roof Plank, grooved for hard wood splines
Fixed Wooden Sashes Double Glazed
Yellow Pine Beams 8 to 9 ft. o.c.
Tension Rod
Abt. 20 ft. apart
Beam Hangers bolted through girders & joint bolts for beams
Girder
Wood Columns
Floor Beams Abt. 8 ft. on C.
Cast-Iron Post Cap
Wood Columns
Cast-Iron Pier Cap
Concrete Basement Floor
1 in. maple top floor
1 in. rough intermediate
4 in. plank grooved for hard wood splines

Fig. 16. Saw-tooth Roof for Textile Weave-shed

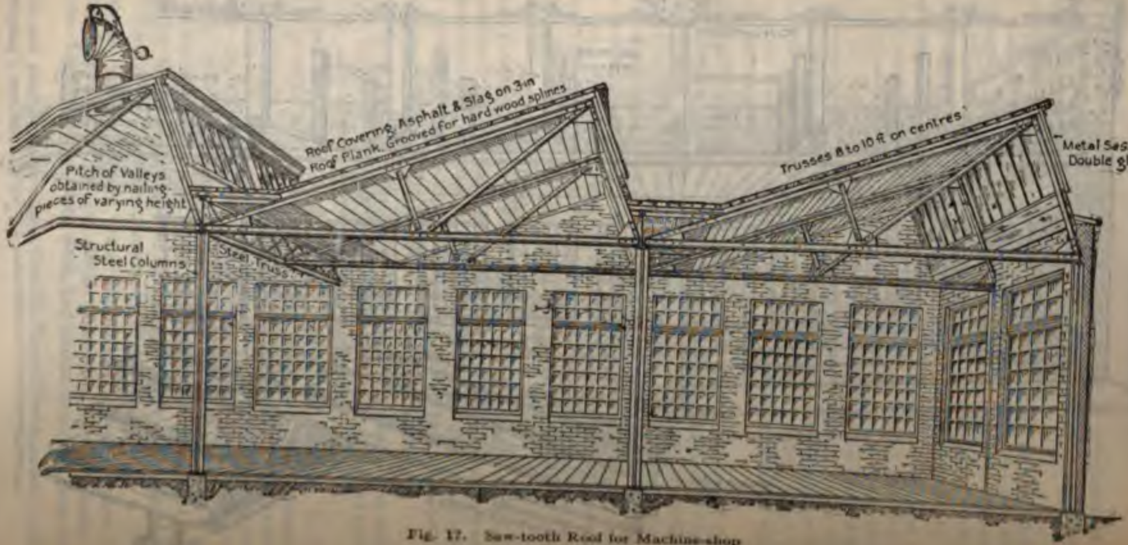


Fig. 17. Saw-tooth Roof for Machine-shop

, and to some dye-houses where careful matching of colors is

of Saw-Tooth Roofs. While the testimony of those who nce with SAW-TOOTH ROOFS is almost uniformly favorable, ave been experienced, practically all of which may be summed her faulty design or poor workmanship. The difficulties in d by

to severe conditions during winter in our northern climates.

lation.

Heat when roofs are thin.

Condensation on the underside of roof and glass when the de is low and there is considerable moisture in the rooms.

hods of Construction. The following suggestions show how ntioned may be obviated if the APPROVED METHODS are applied competent engineers or architects. What is good ENGINEER--point of the manufacturer can also be good FIRE-PROTECTION l any design should be adapted to both if the best interests er are to be served:

ndirect Sunlight. As it is desirable to avoid direct sun-same time obtain an abundance of light, perfectly diffused, ould face approximately north and the glass should be inclined take advantage of the brighter light in the upper sky and to the light by the saw-tooth immediately in front; and, above IFFUSION OF THE LIGHT over the floor rather than on the under lanking.

lass. For the glass an angle of from 20° to 25° from the gle of approximately 90° at the top of the SAW-TOOTH will be ariations depending upon the amount of light required and the er angle at the top is not needed, as it increases the cost, and to be covered and larger spans; more glass, also, is required l the light is not as good, as more light from the sky is lost t is thrown on the under side of the roof.

etails. DOUBLE GLAZING with a space left between the referred on account of its conducting qualities; but it is not except in the more northerly countries. The inside glazing ith factory-ribbed glass, set with the ribs vertical and facing by trusses are then almost unnoticeable.

nd Conductors. CONDENSATION-GUTTERS are needed inside, the sashes, and they should be drained through INSIDE CON-to the outside under the bottom of the sashes, as these latter l are liable to freeze.

ween the SAW-TEETH should be flat, from 14 in to 2 ft in width per ft towards the conductors, which should be of ample size, r 50 ft apart, and preferably less. The necessary PITCH may ss-pieces of varying heights set on top of the trusses, and thus paces.

of Leaks. LEAKS, which are common faults, may ordi-d by a careful design *of the gutters, valleys and sashes,* and by *workmanship and materials. The roof-covering of asphalt ontinuous through the valleys and extend up to the glass,*

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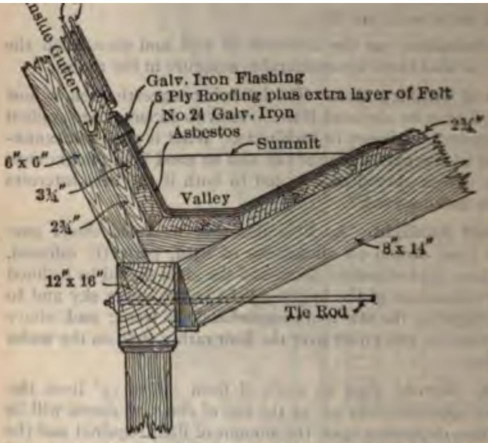


Fig. 18. Detail of Valley of Saw-tooth Roof

greater. Ventilation and cooling of these large areas with c
stories must not be neglected. Ample vents are needed a
form of large metal ventilators with double walls and tight
are recommended in place of pivoted or swinging sash, whic
in driving storms, and when open, allow dirt to blow in
Good windows are advised in side walls and experience has sho

(8) **Details of Framing and Construction.** The FRAM
TEETH may be of timber, steel or reinforced concrete. The
such as will obstruct the light as little as possible, strong en
snow without sagging, and stiff enough to carry shafting m

9. Mill-Construction as Applied to Warehouses

ring to the increasing cost of heavy timbers for wooden construc-
ower cost of the so-called FIRE-PROOF CONSTRUCTION, and also to the
RESISTING qualities of the latter, owners, architects and builders
lly compare the cost of construction, and also the cost of insur-
wo types, before deciding on the one to be used. The difference in
nstruction between these two types is so small, that in many local-
r cost will be in favor of the REINFORCED CONCRETE or other type
F CONSTRUCTION. The cost of construction is also in favor of the
YPE, where both long spans and strength are required.

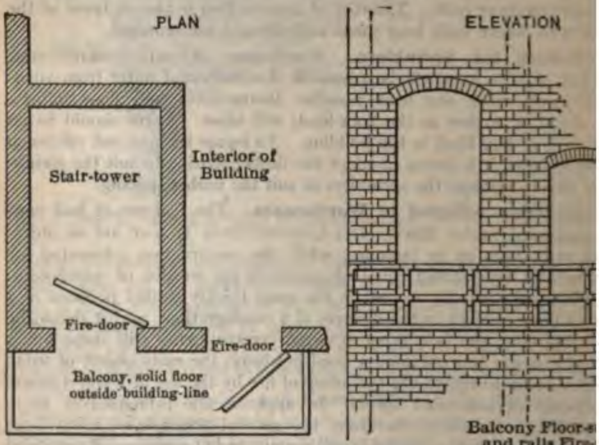
pacing for Sprinklers. Warehouses of MILL-CONSTRUCTION
ilt so as to allow the best possible distribution of water from AUTO-
KLERS, with the least possible obstructions, and floor-timbers,
ould be as few as the floor-loads will allow. There should be no
aces of any kind in the building. To insure the greatest efficiency
systems, it is better to adapt the timber-spacing to suit the sprink-
han to arrange the sprinklers to suit the timber-spacing.

truction Adapted to Warehouses. The features of bad con-
ntioned under WHAT MILL-CONSTRUCTION IS NOT are as objec-
varehouses as in factories, while the construction advocated for
e used with almost equal advantage in the erection of warehouses.
atter are usually erected in the more thickly settled portions of a
e more subject to the dangers of a conflagration; and it should be
hat even the best SLOW-BURNING CONSTRUCTION will stand but a
ter a fire has obtained a good headway, the main object of MILL-
N being to retard the spreading of fire by the use of heavy timbers
ence of concealed spaces. In applying the principles of MILL-
N to warehouses, therefore, the general principle of using large
d as far apart as the loads will permit, and of avoiding all concealed
d be constantly kept in mind.

se-Floors, however, are generally required to sustain heavier loads

separate sections by fire-walls, thus reducing the liability to one fire,
ing an opportunity of storing hazardous goods in one or more
non-hazardous or less hazardous goods in the remaining
ground is available, it is better to have a building of LARGE AREA
HEIGHT divided into fire-sections, than to have a building of LESSER
GREATER HEIGHT, as the former construction affords a more
of goods, and less concentration of values. Buildings of this type
limited to 65 ft in height, and to six stories, thus discouraging the
of floors. Piled goods should be kept at least 18 in away from
allowing for the distribution of water from the sprinklers.

Walls should be of brick, and not less than 13 in thick in the
and they should be increased in thickness on the lower floors to
additional loads. PARTY WALLS should be increased at least 4 in in
and all walls should be laid in cement mortar, should extend above
at least 36 in and be coped with stone, salt-glazed terra-cotta, or
combustible materials. OPENINGS IN DIVISION WALLS should be
few as possible, not over three in each story, they should not exceed
each in area, and should be protected by double, automatic, sliding
as specified elsewhere. (See Chapter XXIII, page 907.)



Note: Walls of brick or other approved material, built solidly
from foundations to at least 36 inches above roof.
Stair-treads, etc., of fire-proof material.

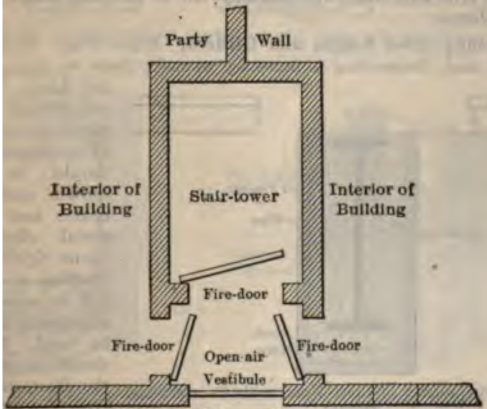
Fig. 19. Tower Fire-escape. Outside-balcony Entrance

Openings in Walls. As a protection against fires from
erties, OPENINGS IN OUTER WALLS should be small, limited to as few
and protected by standard fire-shutters and doors, or standard
windows. If the surrounding buildings are of hazardous occupancy
construction, and the distance between the warehouse and the latter
feet, shutters are preferable, as wire-glass windows are
where the exposures are moderate. Even though the building

other buildings, the protection of WINDOW-OPENINGS may prevent f fire from story to story through the windows.

and Beams which support the floors and roof should be SINGLE less than 6 in in least dimension, and with a sectional area of not sq in; while columns should be not less than 8 by 8 in in cross- he upper story, and should be increased in size in the other stories of any additional loads. The beams and girders should be SELF- Fig. 2), and the floors should be built as outlined under STANDARD CTION, page 760, inclined at least 1 in in 20 ft, made as nearly as possible, and scuppered to the outside of the building. These ould be set in brick-work at frequent intervals, of sufficient size to e maximum amount of water from each floor, and so constructed ll prevent the admission of cold air to the building. (See Fig. 11.)

The floors should be continuous from wall to wall, avoiding holes irways, elevators, etc. All such openings should be enclosed in a or in TOWERS extending not less than 36 in above the roof, coped l accessible from each story by means of an outside balcony (Fig. 19).



Openings in face-wall extend from floor to ceiling, Vestibule-floor of fire-proof construction, Railing at opening.

Fig. 20. Tower Fire-escape for Adjoining Buildings

impossible, owing to the location or otherwise, to have these open- outside, they should be placed in BRICK TOWERS constructed inside and connecting with an entrance to a fire-proof vestibule, open to . There should be openings from each story to the vestibule, each y standard fire-doors (Fig. 20).

Tanks for Automatic Sprinklers are usually placed on extensions ers, and they should be built to carry the additional load imposed. to the roof of the building may be had from a window or windows ne tower, and such opening or openings *should be protected by fire- pecially where the tower is elevated a sufficient distance to allow be placed inside of the tower, thus preventing flames from gaining tower and destroying the tank and tank-supports.*

Boilers should be, preferably, in a separate building, cut off by
fire-doors from the warehouse; or, if in the main building, should be
a room of FIRE-PROOF CONSTRUCTION, access to which should be fr
the building only.

Structural Steel Members should never be used in this type of c
as they will not resist even a moderate fire. If used, they should be
with fire-proof material. The lintels should be brick arches and
sections.

10. Steel and Iron Structural Members in Warehouse-Con

Metal versus Wooden Standard Members. Owing to the f
beam or column of STEEL or WROUGHT IRON when heated will fail by
or bending very much sooner than an equivalent beam or post of w
important that such members be of WOOD, provided that the woo
have a sectional area of at least 72 sq in, and are not less than 6 in in le
sion, and that WOODEN COLUMNS have a sectional area of not less than
CAST-IRON COLUMNS, also, will generally fail in fire and water so
wooden columns.

Fireproofing Steel Beams and Girders. When STEEL BEAMS
UMNS are used, fireproofing is necessary to make them as FIRE-RE
the floors. Su
and girders may
PROOFED as sho
21. Metal-w
should be p
shown, and ti
beams and gi
metal clips;
insure rigidity
pouring of the
and to keep th
alignment, fo
be used. The
should be pou
the floors are
after the woo
are in positi
completion, the insulation should be at least 1 in at the ed
flanges, 2 in under the lower flange of the beam and 3 in under the l
of the girder. The webs should be filled solid. Where there is lit
of a combustible nature in the building, the beams may be protect
in Fig. 22. (See, also, pages 863 to 866.)

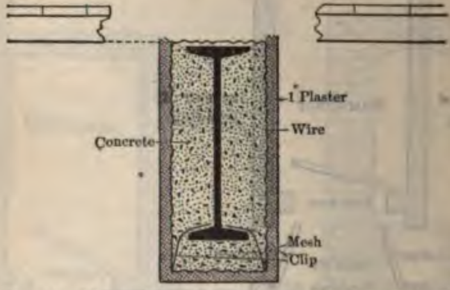
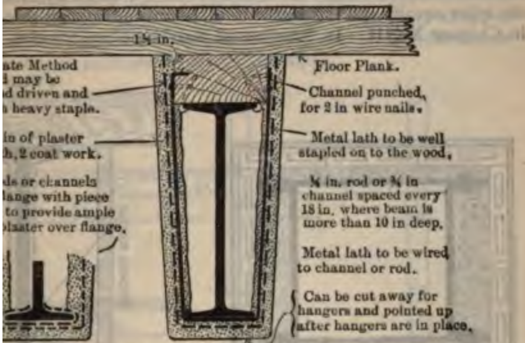


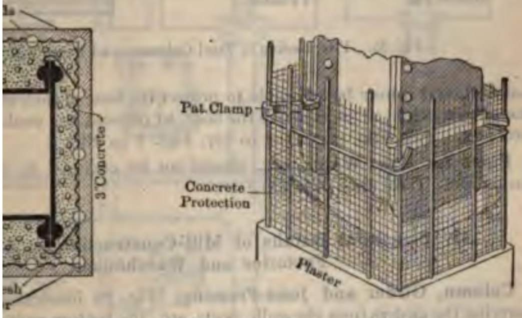
Fig. 21. Fireproofing of Steel Beam with Concrete and Plaster

Fireproofing Metal Columns. COLUMNS, either STEEL, WRO
or CAST-IRON, should be protected even to a greater extent than girders
and should have at least 3 in of concrete at the flanges, at least
edges, and be filled solidly against the webs. Fig. 23 shows two column
by concrete held by wire mesh on ⅝-in rods, and all securely held to
by metal clips. Forms should be used and the concrete should be
the girders and beams are protected. Steel beams, girders and
difficult to protect, especially at the intersections of steel and woo
insulating material can best be applied before the floors are laid.
proofing of these members will be of little avail, unless the material

ıetal members, and applied by workmen who understand such


1⅝ in.
ate Method
l may be
d driven and
ı heavy staple.
in of plaster
h, 2 coat work.
ls or channels
lange with piece
to provide ample
plaster over flange.
Floor Plank.
Channel punched, for 2 in wire nails.
Metal lath to be well stapled on to the wood,
⅜ in. rod or ¾ in channel spaced every 18 in. where beam is more than 10 in deep.
Metal lath to be wired to channel or rod.
Can be cut away for hangers and pointed up after hangers are in place.

Fireproofing of Steel Beam with Metal-lath and Plaster



Fireproofing of Steel Columns with Concrete and Plaster

tes the PROTECTION OF A ROUND COLUMN by reinforced concrete. ɛe is held in position ı metal furring, held metal clips or ties. should be at least 4 ms should be used in olumns. In addition ıforcements for these reinforcement should ns of iron rods wound ıesh, and placed 12 in er the forms are re- ooden floors are laid, l girders should be -in thickness of hard interstices between the woodwork *and the insulation.* Tile,

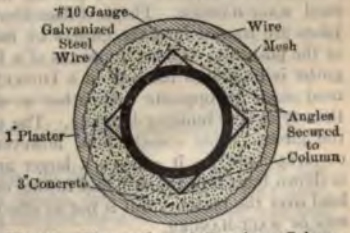


Fig. 24. Fireproofing of Cast-iron Column with Concrete and Plaster

owing to the difficulty of properly bonding it, is not as effective
but if securely bonded by means of metal, it is quite satisfactory.
illustrates the PROTECTION OF A GIRDER AND A COLUMN by means of
are other equally efficient methods of beam and column-protection,
in Chapter XXIII. In buildings of warehouse-construction, heavy

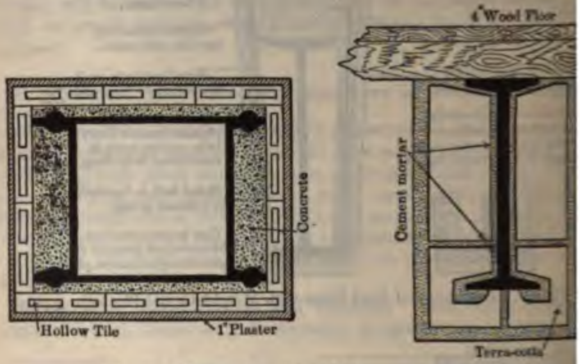


Fig. 25. Fireproofing of Steel Columns and Beam with Tile

handled, and it may be advisable to protect the base of each column
metal to a height of 36 in above the floor, to prevent any weakening
proofing. (See, also, pages 822 to 827, Figs. 1 to 13.)

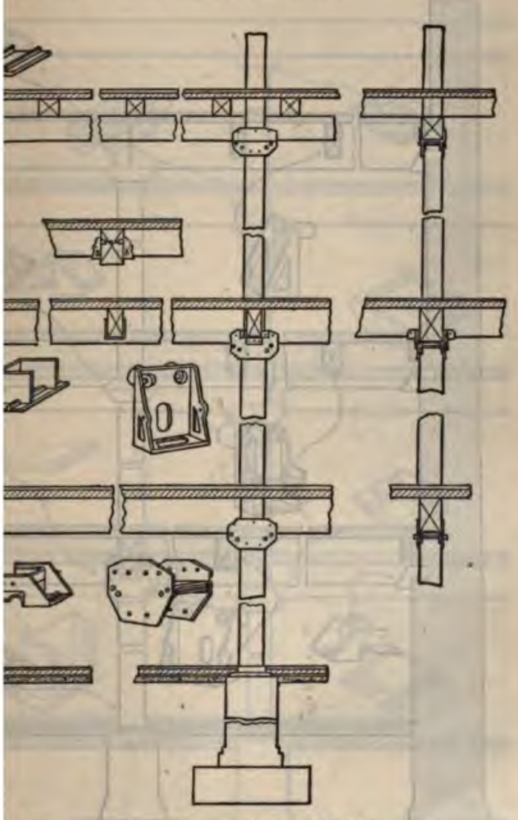
Pipes for Gas, Water, etc., should not be enclosed in column
insulation. (See, also, page 827.)

11. Structural Details of Mill-Construction as Applied Factories and Warehouses

Column, Girder and Joist-Framing. Fig. 26 illustrates the
carrying the girders from the walls, posts, etc., the bottom post resting
POST-BASE. The first floor above the basement is shown with
girders only, and heavy mill-flooring set on them. The girders are
the post in a steel POST-CAP, and are hung clear of the wall in
steel WALL-HANGER. The next floor above shows the construction
joists are framed into the girders by means of JOIST-HANGERS.
at the post, also, is done by means of a DUPLEX FOUR-WAY
girder is built into the wall in a DUPLEX WALL-BOX. The
used singly or opposite each other as required and are bolted
thus tying the building laterally. The upper floor shows the joists
the girder. This construction, however, does not conform to
CONSTRUCTION, as it exposes a larger amount of timber-surface.
is shown built into the wall and resting on a WALL-PLATE. This
load over the masonry but is not as effective in preventing dry-rot
BOX or WALL-HANGER.

Steel and Malleable-Iron Post-Caps and Bases. Fig. 27
details of construction which may be used. The bottom post
POST-BASE. The POST-CAP shown on the bottom post is a

while the POST-CAP above it is one of the malleable-iron type, National Board of Fire Underwriters. The POST-CAP shown is of malleable iron and intended for lighter construction or run across the post as shown. The girders in every case are he wall by means of approved WALL-HANGERS and the beams girders in malleable-iron JOIST-HANGERS.



Mill-construction. Column, Girder and Joist-framing

t-Caps and Bases. Fig. 28 illustrates other details of con- owest post rests on a heavy, cast-iron, ribbed POST-BASE. r-girders are carried at the post by means of heavy, cast-iron e built into the wall in cast-iron *WALL-BOXES. When cast* *OST-CAPS it is essential that it be made extra-heavy, as cast* *in on account of the uneven shrinkage when cooling, which* *stresses and weakens the caps. Flaws, also, may develop*

during the manufacture which weaken the caps and greatly impai
of the building. An objection to cast iron is its tendency to crac
during a fire when cold water is thrown on it. The POST-CAPS show
are of cast iron for the first and second floors, Duplex steel for the
and malleable iron on the top post.

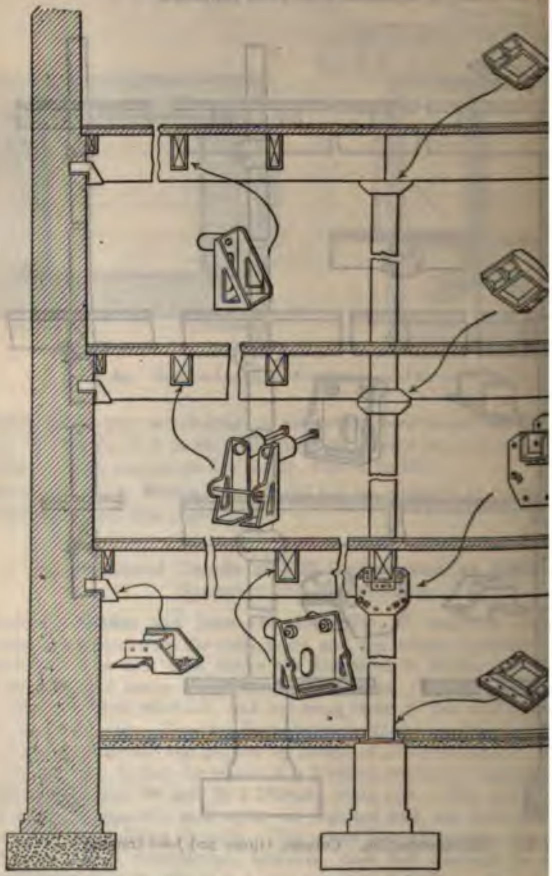
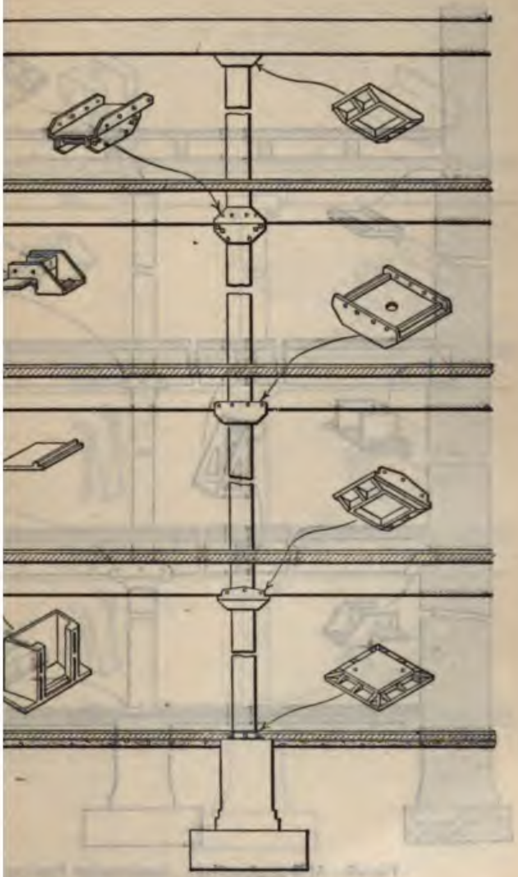


Fig. 27. Mill-construction. Malleable-iron Post-caps and Ba

Duplex, Combination Post-Cap. Fig. 29 illustrates the use o
COMBINATION POST-CAP on the bottom post. This cap is made with
iron lower part and a steel upper part. The POST-CAP shown
post is called the IDEAL POST-CAP and consists of a steel upper
angles riveted underneath to fit the post. The cap shown

ast-iron cap. The WALL-HANGER, WALL-BOX, WALL-PLATE shown are used in STANDARD CONSTRUCTION.



. Mill-construction. Cast-iron Post-caps and Bases

ps. Fig. 30 illustrates various forms of steel POST-CAPS. *P is shown on the bottom post and the* VAN DORN POST-CAP *on . On the top post the* STAR POST-CAP *is shown.* This has *op of the post must be slotted to receive it.* Steel JOIST-

HANGERS are shown for the two lower floors. The IDEAL illustrated in the lower floor. It is spiked to the sides and top The VAN DORN JOIST-HANGER is shown in the second floor, whil STIRRUP is shown in the top floor. The WALL-HANGERS illustra approved type.

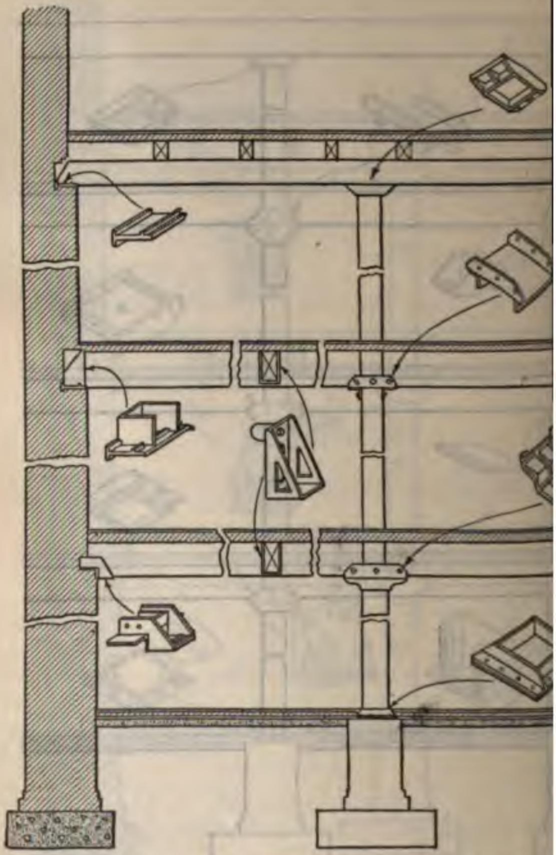
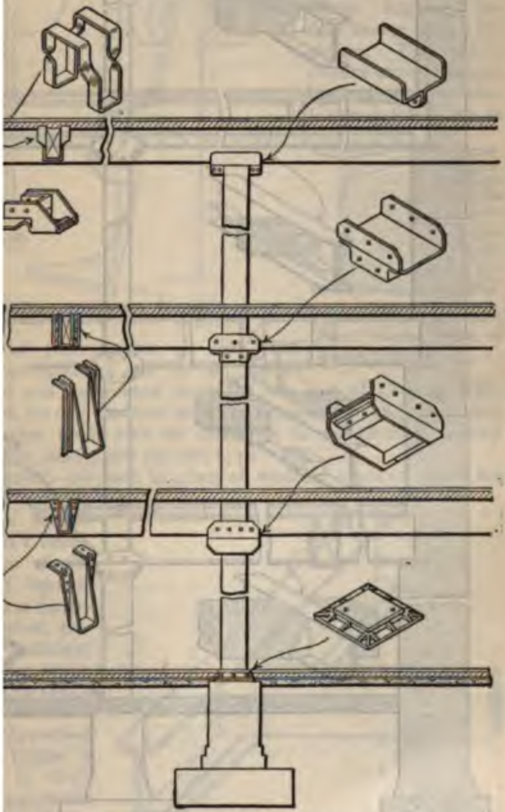


Fig. 29. Mill-construction. Combination Post-caps, etc

Framing Steel Beams and Girders. Fig. 31 illustrates the GIRDERS in place of WOODEN GIRDERS and their connections with In this kind of construction it is necessary to fireproof the steel are more readily affected by heat in case of fire than large Intense heat often causes them to collapse and ruin a building.

ɔor is used where the **I** beams and wooden beams are of the
HANGER provides an extra bearing for the timber and has
tory. The HANGER shown in the second floor is used when
ise the wooden beam above the lower flange of the steel
R brings all the load on the lower flange of the **I** beam and



g. 30. Mill-construction. Steel Post-caps, etc.

ıge for the wooden beam. It is used singly or in pairs on
red, and is bolted through the web of the **I** beam. This has
very economical and efficient construction. In the third
eam is shown framed to the **I** beam by means of a *SHELF-*
form *of construction it is necessary to rivet the SHELF-*

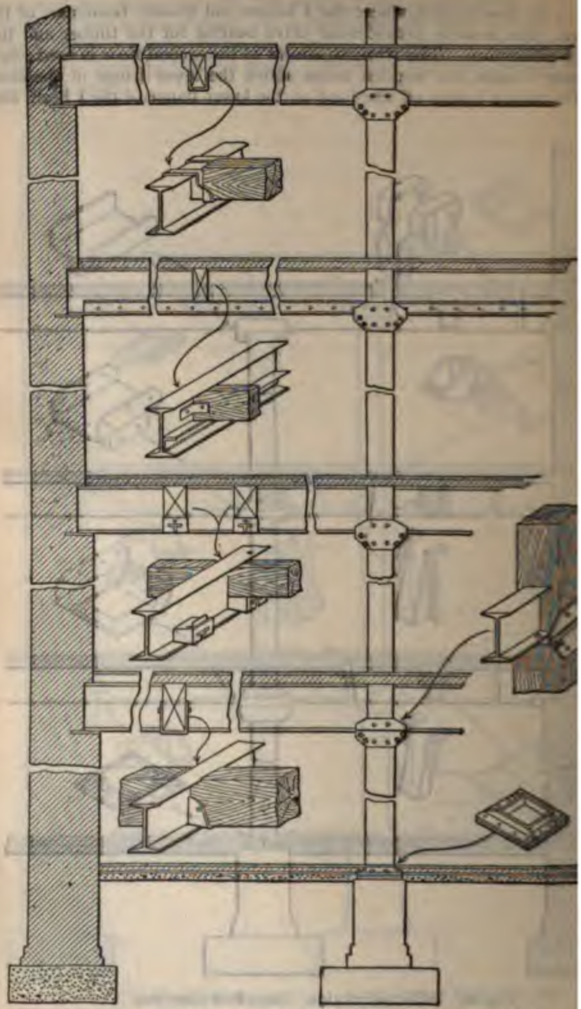
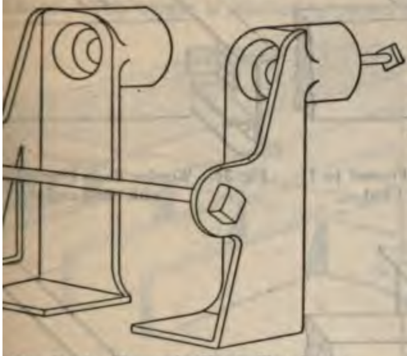


Fig. 31. Mill-construction. Framing Steel Beams and Girders

ANGLE to the web of the I beam. The upper detail shows the old STIRRUP passing over the top flange of the I beam and carrying th beam. The POST-CAPS shown are the DUPLEX STEEL POST-CAPS w approved by the National Board of Fire Underwriters.

12. Connections of Floor-Beams and Girders

-Hangers and Joist-Hangers. To render the construction, and rly the girders, SLOW-BURNING, it is important to have no hollow spaces the top of the girders and the flooring, that is, to have the top surface of the floor-beams flush with that of the girders. This, of course, necessitates framing the floor-beams into the girders. For HEAVY CONSTRUCTION the only kind of framing that is permissible is one in some kind of JOIST-HANGER is used. The kinds of JOIST-HANGERS now in the market en illustrated and commented on in the last part of Chapter XXI. e floor-beams are 6 by 12 in or larger in cross-section, and the girders ood, the author would give the preference to the DUPLEX HANGER Fig. 32. (See, also, pages 752 and 753.)



. 32. Duplex Hanger for Heavy Floor-beams

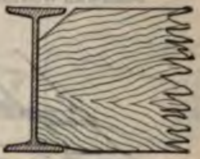


Fig. 33. Framing I Beam and Wooden Beam of Same Depth

EL-BEAM GIRDERS are used in place of WOODEN GIRDERS, there are methods in use for the wooden beams. shows a steel **I** beam, wooden beam of the epth framed into it ng on its lower flange. cases, however, this afford a sufficient for the wooden beam. shows a SHELF-ANGLE to the web of the **I** Whenever this method orting the beams is ough bolts or rivets be used to support the rried by the SHELF- Each ¾-in bolt may dered to support 3 000 ch side of the girder, h ⅞-in bolt, 4 000 lb. thods shown in Figs. 35 and 36 are sometimes used, but are open to n on account of the weakening of the wooden beams when loaded. shows a STIRRUP-TYPE of *hanger. This construction* permits the

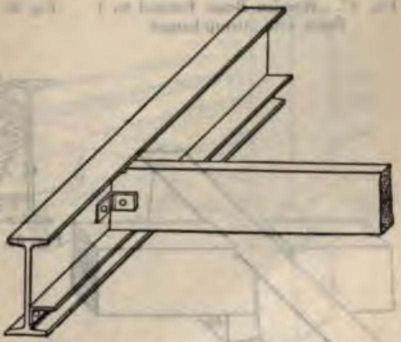


Fig. 34. Wooden Beam Framed to I Beam with Shelf-angle

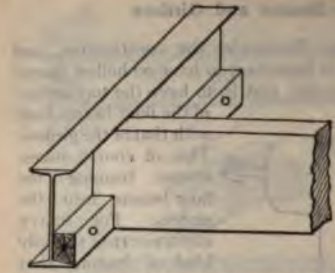


Fig. 35. Wooden Beam Framed to I Beam with Wooden Cleat

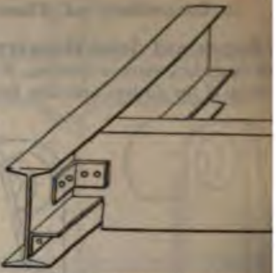


Fig. 36. Wooden Beam Framed
with Shelf-angle

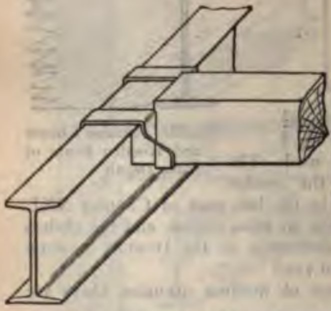


Fig. 37. Wooden Beam Framed to I Beam with Stirrup-hanger

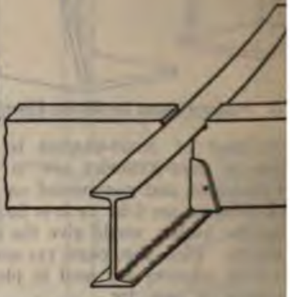


Fig. 38. Wooden Beam Framed
with Duplex Hanger

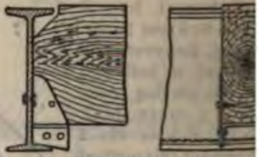
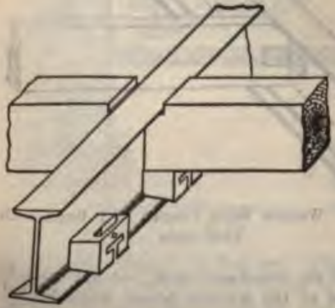


Fig. 39. Wooden Beam Framed
with Duplex Shelf-hang



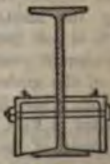
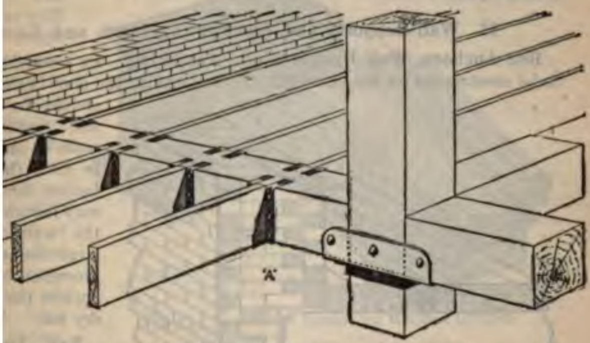


Fig. 40. Wooden Beam Framed to I Beam with Duplex Box-hanger

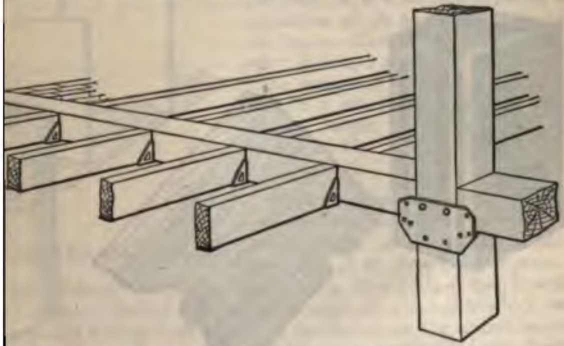
e wooden beam at any desired height, and has proved satisfactory. s can be used with any depth of beam or girder, and are furnished icturers of steel JOIST-HANGERS of the various types, as well as by vho can make WROUGHT-IRON STIRRUPS. Fig. 38 shows the Du- HANGER for framing a wooden beam flush with the lower flange 1. This hanger is attached by means of bolts. Fig. 39 shows



g. 41. Floor-framing with Van Dorn Hangers and Post-caps

sign of HANGER, with the SHELF-CONSTRUCTION used to carry beams up to 4 in above the lower flange of the **I** beam. Fig. ANGER for carrying the wooden beams 4 in or more above the f the **I** beam.

RS described in Figs. 38, 39 and 40 are all of the DUPLEX TYPE, nstructed that all the load is carried on the lower flange of the



ig. 42. Floor-framing with Duplex Hangers and Post-caps

1 is a very satisfactory and ideal construction whenever it is ame wooden beams into and not rest them on the **I** beams. The y economical one for framing wooden beams *to **I** beams, as the* hing these *HANGERS can be punched while the steel is being fabri-*

cated, and the HANGERS are attached to the steel beams by means of
the wooden beams are put in place. These HANGERS are provide
or lag-screws for anchoring the wooden beams securely to the st
Fig. 41 shows a floor-framing with the VAN DORN STEEL HANGERS.
shows the floor framed with the DUPLEX TYPE OF HANGER and POST-
same principle of construction is applicable to larger wooden bea
farther apart.

13. Wall-Supports and Anchors for Joists and Girder

Box Anchors, Wall-Hangers, etc. Anchoring. In a warehous
to be constructed on the SLOW-BURNING PRINCIPLE, the floor-beams a
should be an
and supporte
walls in such a
in case the b
burned through
may fall witho
the walls; a
large timbers
provision shoul
against the pos
dry rot.

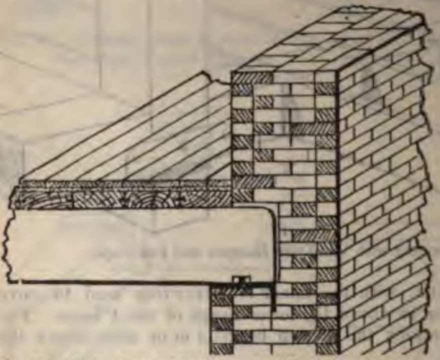


Fig. 43. Early Form of Beam-support in Mill-construction

Box Ancho
method of supp
beams in MILL-C
TION as origi
veloped in the
land mills is
Fig. 43. Thi
the requireme
mentioned, but it weakened the walls to some extent. The GOETZ

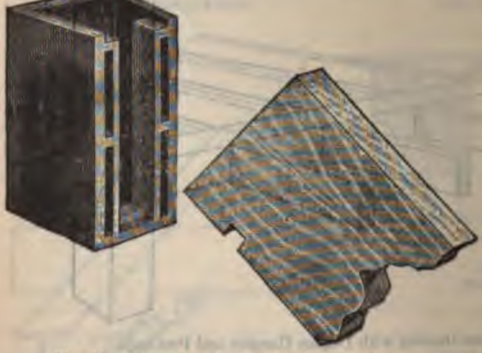


Fig. 44. Goetz Box Anchor for Wooden Beams

Fig. 45. Goe
chor for Woo

BOX ANCHORS shown in Figs. 44, 45 and 46 and the DUPLEX WALL-H
in Fig. 47 are decided improvements on the anchor shown in Fig. 4

advantages of the latter without weakening the walls, unless the
re very wide. The WALL-BOX as shown in Fig. 47 is made
ble-iron bottom plate and a steel box above. It has a rib
t the back, which extends up and down, and acts as a secure
the brickwork. These WALL-BOXES are made wedge-shape,
erefore
pull
e wall.
weight
beam,
will be
holds
he box
to the

fire or
joists
ugh or
falling
them-
he an-
leave
nding.

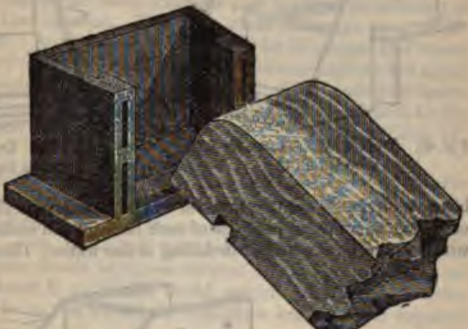


Fig. 46. Goetz Box Anchor for Wooden Girders

not even weakened by the space left in it, because the box
the crushing strength of this CAST-IRON BOX is much greater than
all. No break or breach is made in the wall, and the box that
ely held, forms a space for the easy replacement of the wooden
box provides a perfect and secure foundation for each beam.
efective flue cannot ignite a beam-end, because it is protected by
a ventilated, CAST-IRON BOX. The WALL-BOXES have air-spaces, also, in the sides, ½ in wide, which permit a circulation of air around the ends of the beams, effectually preventing dry rot. If timber is wet or unseasoned these wall-boxes allow it to dry out after it is put in the building. The average weight of a box like that shown in Fig. 45, for 2 by 12-in joists, is 10 lb.

x Wall-box with Ribbed Plate

Wall-Hangers. Another device for obtaining the same results in a different way is the WALL-HANGER. Figs. 48 and 49 show DUPLEX WALL-
rge timbers. The hanger shown in Fig. 49 is made of open-hearth
ra-heavy. Each of these hangers is provided with a plate which has
g on the wall, and the bearing of the timbers on the hanger is also
ms not exceeding 10 in in breadth there is probably little choice
BOX ANCHOR, Fig. 46, and the WALL-HANGERS, Figs. 48 and 49,
s in the price and appearance. When the WALL-HANGER is used,
in the wall, and a saving of 6 in in the length of the beams is
h in some cases would be a consideration. For girders 12 by 14 in
in cross-section; the author *believes that the hanger shown in*

Fig. 48. Duplex Wall-hanger for Large Wooden Girder

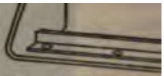


Fig. 49. Duplex
hanger for Large

boxes is obviously greatly superior to the ordinary method of
or girders to walls, and the use of such hangers will undoubted
which would be caused by the falling of the walls. These are

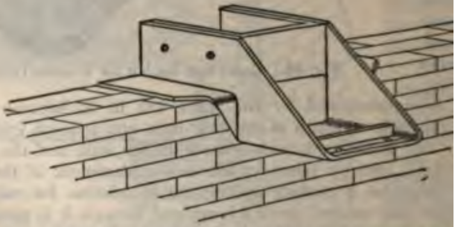


Fig. 50. Application of Wall-hanger to Brick Wal

pulled down by the ORDINARY IRON ANCHORS when the bea
shows the application of a WALL-HANGER.

14. Weakness of Wrought-Iron Stirrups when Exp

Stirrups and Fire-Tests. Referring to this subject, Profes
of Washington University, said: " The recent fire-tests of st
brick walls which were made under my supervision in this
show very conclusively that unprotected stirrups are extr
These stirrups become red-hot in a few minutes and then
burn away the ends of the beams; and they also bend down,

15. Post and Girder-Connections

-Plates, Wooden Bolsters, etc. Whenever a building is con- ı wooden posts extending through several stories, each upper post n an IRON CAP-PLATE, fitted over the post below, and never on a n on a WOODEN BOLSTER. A BOLSTER would not be objectionable or the fact that the pressure under the post is generally sufficient to ers of any kind of wood. Then, too, there is always some settle- m shrinkage. As posts are used expressly for the support of beams e IRON CAPS must, of course, extend sufficiently beyond the upper d ample bearing for the end of the girder. This bearing in square l be equal to at least one-half the load on the girder divided by the e of the wood to crushing across the grain, as given in Table IV,

A 12 by 14-in yellow pine girder is designated to support a possible o lb. What bearing should it have at the ends?

The safe resistance given for long-leaf yellow pine to crushing across 350 lb per sq. in. One-half the load on the girder is 19 000 lb, e bearing area should be 19 000 divided by 350 or about 54 sq in. th of the beam is 12 in this would require a bearing lengthwise of 4½ in. In no case should the bearing be less than that required rule.

16. Form and Material of Post-Caps

versus Steel Post-Caps. Formerly CAST-IRON POST-CAPS were framing of the girders at the columns and posts. But the uncer- ed to the use of cast iron, and the necessity of extremely heavy e safe construction have led most engineers to specify STEEL POST- are unquestionably the strongest form of construction for fram- girders. The use of STEEL POST-CAPS is to be recommended, there ertainty regarding the strength of steel as there is concerning the ast iron used for post-cap construction. Internal stresses due to ng may seriously affect the strength of a CAST-IRON CAP, while a l casting may be used, undetected, and affect the safe carrying that failure of the cap may occur even from the vibration due to y in the building.

Post-Caps are still used in some localities and a few of the com- s well as those of STEEL POST-CAPS are shown. Fig. 51 shows a s frequently used for light construction. Fig. 52 shows a similar ndrical post. These caps permit the use of girders wider than the the girders and floor-beams are in place, and especially when the ccupied, there is no danger of the girders or posts slipping on fact it would require a greater force to move them. The girders d together longitudinally by IRON STRAPS spiked to their sides. s, however, consider it important in a building of SLOW-BURNING r, to have the posts tied together in vertical lines, and the girders ch a way that they will be self-releasing without pulling down igs. 53 and 54 show two POST-CAPS which fulfill these requirements. aps the ends of the girders are not fastened by bolts or spikes, but lace and tied longitudinally by means of the LUG L on the GOETZ PINS on the DUVINAGE CAP; so that in case the girder is burned to point, it can fall without pulling on the post. Provision is also ting the cap to the upper post. The *author doubts very much,*

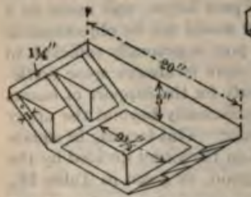


Fig. 51. Cast-iron Post-cap for Square-section Wooden Post

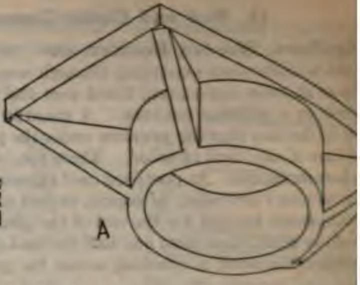


Fig. 52. Cast-iron Post-cap for Cylindrical Post

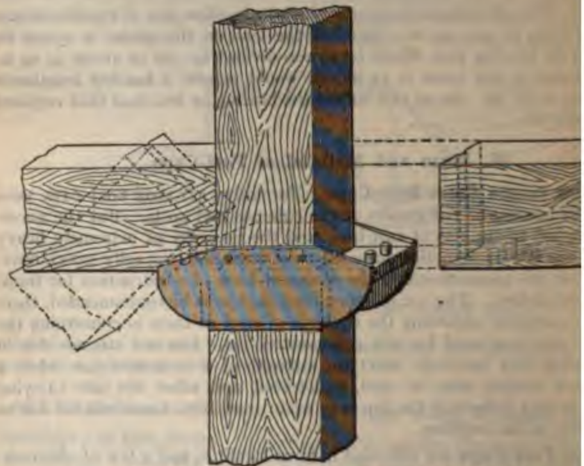


Fig. 53. Cast-iron Duvinage Post-cap with Beam-pins

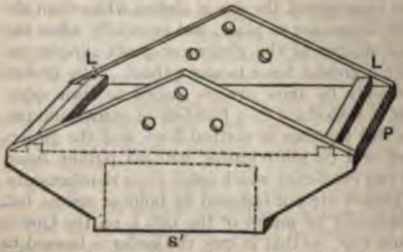


Fig. 54. Cast-iron Goetz Post-cap with Beam-lugs

Fig. 55. Cast-iron Po High Sides

, if posts bolted together in this way will stand after the girders have s the planking will be likely to pull the posts over, even if they do not quickly as the beams. Fig. 55 shows another form of CAST CAP with es, allowing lag-screws to be driven in the holes to tie the girders.

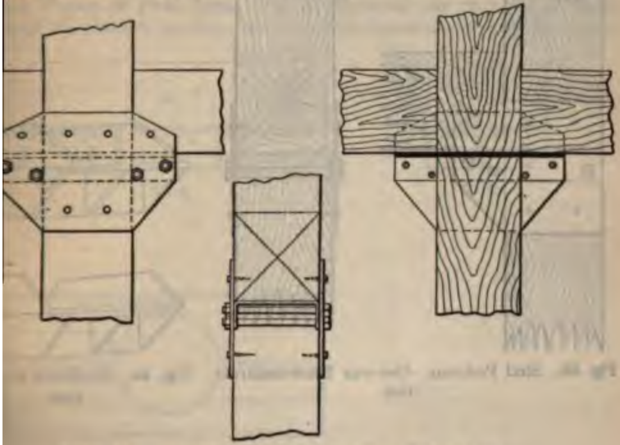
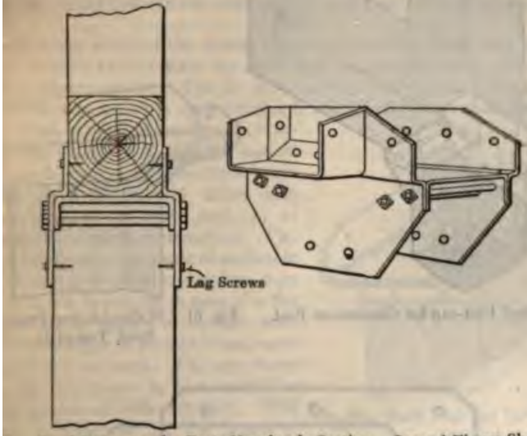


Fig. 56. Steel Post-cap with Side-plates and Brackets



7. Steel Post-caps for Posts Varying in Section. Second Figure Shows Four-way Beam-construction

eel Post-Cap, which is approved by the National Board of Fire Under- and bears their label, is shown in Fig. 56. This POST-CAP is made up of de-plates and heavy steel brackets, all held rigidly *together by means* heavy bolts. The *posts and girders are fastened to the cap by* means of

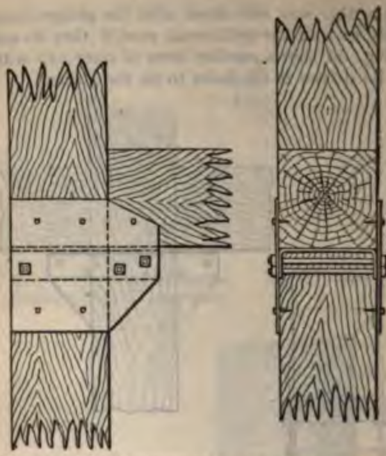


Fig. 58. Steel Post-cap. One-way Beam-construction

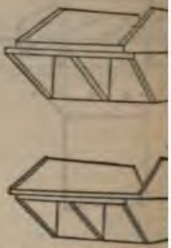


Fig. 59. Malleable- caps

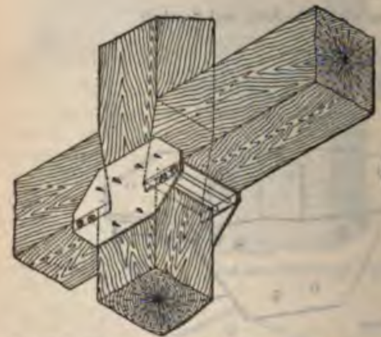


Fig. 60. Steel Post-cap for Continuous Post

Fig. 61. Malleable-iron Steel Top-plate

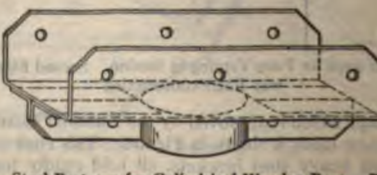
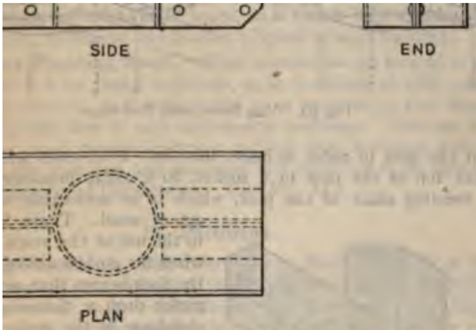


Fig. 62. Steel Post-cap for Cylindrical Wooden Post. Perspective



Steel Post-cap for Cylindrical Wooden Post. Elevations and Plan

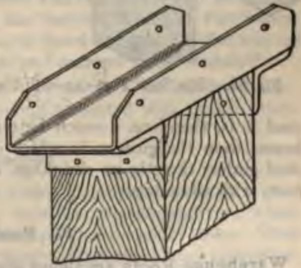


Fig. 64. Steel Post-cap for Light Construction

he width of the girder is greater than that of the post below. In
R-WAY BRACKETS are riveted to the side-plates to provide for the
ISTRUCTION. Fig. 58 shows a ONE-WAY CONSTRUCTION. Fig. 60
CAP which is used when it is required to run a post through two
is what is known as a CON-
-CAP. The bracket instead
clear across the cap is made
h sides and fitted into
ched into the post, so as
re rigid construction. Fig.
POST-CAPS made of malleable
preferable to cast-iron caps
ot break off in case of a fire
ter comes in contact with
anger is present when CAST-
s are used. The cap shown
vo parts so that it will fit
ers of different sizes. This
pproved by the Board of
ters. Fig. 61 shows a COM-
-CAP, the upper part of which is made of steel plate, and the
malleable iron. Figs. 62 and 63 show STEEL POST-CAPS FOR
They are also frequently used for pipe-columns and concrete-
(See, also, Steel-Pipe Columns, page 469 and Lally *Columns,*
477.) Fig. 64 *shows a STEEL POST-CAP intended for lighter*

construction. Fig. 65 shows VAN DORN POST-CAPS. Fig. 66 illus STAR POST-CAP which is made of a bent steel plate with a fin projecti into a slot in the post. Both are approved by the Underwriters. It is

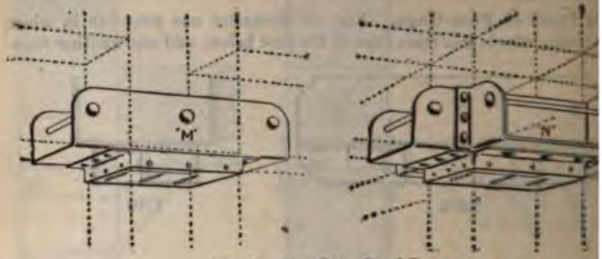


Fig. 65. Van Dorn Steel Post-caps

to slot out the post in order to insert this fin. POST-CAPS which encircle the top of the post in a socket, to a great measure te vent the twisting effect of the post, which is so noticeable whe are of wood. There is a to the use of the FOUR-WAY when the girders are of wo the floor-beams that are hu girder drop a distance equ shrinkage in the girder, if are hung in stirrups, or b this amount if they are DUPLEX HANGERS. The b ported on the POST-CAP ca at all, and consequently the be higher over the beam by the posts, than over mediate beams. In one where deep beams were unevenness in the floor am nearly an inch and was ve able. Wherever wooden g used it is, therefore, a much better construction to support all of beams from the girders, in which case the shrinkage will be unifo steel girders there is no shrinkage, and a beam may be, placed op posts with advantage.

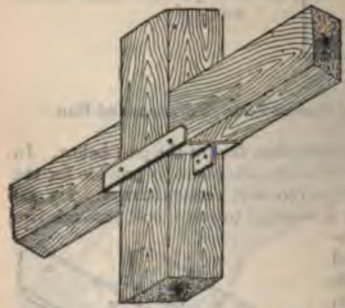


Fig. 66. Star Steel Post-cap with Fin

17. Roofing-Materials

Warehouse-Roofs are almost always flat and, like floors, should b ous from wall to wall, without openings. The occupancy of such calls for little light, and hence skylights and other roof-structures a quired.

Dampness and Leaks. Stored goods may be very easily damaged and roofs, therefore, should be of such construction that they will prev ness, either through leakage or condensation. While roofs are usuall flat as possible, the incline should be sufficient to drain readily, and

oofing will stand hard usage, as is shown by its continued use on
ls and steamers; but it is not adapted to large buildings.

for Flooding Roofs. When warehouses are located in congested
ounded by higher buildings, or by buildings of light construction
occupancy, their roofs should be so constructed that they may be
g severe fires in such surrounding buildings. This can be accom-
ing good roofing-materials, making high flashings, waterproofing
ve the roof-line, and providing roof-outlets of types that will allow
stoppers at the scuppers. (See Fig. 11.)

18. Partitions

ng Partitions. This refers only to those light walls or enclosures
e rooms, etc., and not to those walls which divide the building into
RTITIONS, as here defined, bear no floor-loads. Buildings of the
G TYPE, for occupancies described above, need but few partitions,
ould be built of non-inflammable materials, preferably metal lath
n light, metal studding. All cupboards, closets, lockers, etc., in
this type should be of metal, or other equally non-inflammable

19. Doors and Shutters

rwriters' Specifications. Doors and shutters should be built as
e Rules and Requirements of the National Board of Fire Under-
e construction and installation of fire-doors and fire-shutters, as
tions are accepted by architects and builders as the standard.

nings should be limited to 80 sq ft, or less, each, and all communica-
buildings or sections of a building protected with double, auto-
doors.

20. Fire-Protection

Sprinklers, supplied with an ample quantity of water at a good
needed in mills, storehouses, factories, warehouses, etc., where
oods are made or stored, or where large values are at stake. They

Difficulty of Estimating Costs from Tables. The cost
this type of construction depends upon the cost of material
labor, and as the cost of either varies greatly in different loc
similarly constructed buildings must also vary. Even if the
materials does not vary, the cost of buildings of the same area
upon the height, floor-loads, distance between bearing-points,
it is difficult to deduce a table accurate enough for use in com
approximate cost of buildings per square foot of floor-area
architects † states: "Experience has taught us that estima
a building either by the SQUARE-FOOT METHOD or the CUB
has proved dangerous and misleading, and it was abandon
years ago except to obtain a general idea of the cost of
have found that the only reliable way to approximate the co
is to block it out and to figure the approximate quantities, whi
prices prevailing at the time the building is to be erected, will
mate cost of said building." Owing to the high cost of lumb
building will cost but little, if any, more than a building of MIL
and owing to this fact it is always advisable to determine the
of both types before deciding upon the type to be used. B
CONSTRUCTION are becoming obsolete in some localities, and ow
rate of insurance on buildings of FIRE-PROOF CONSTRUCTION, th
type are much preferred, as in the end they cost less. It is no
compare the TOTAL COST OF LABOR with the cost of the LAB
the cheaper labor is often the more expensive in the end, this d
upon the locality and the conditions imposed. Tables showing
cost of buildings of the MILL-CONSTRUCTION type are comput
of mill-buildings of light construction (cotton-mills with late
and are not adapted to computing the cost of heavy wareh
factory-construction. The figuring of the cost of such building
PER SQUARE FOOT gives, at the very best, only approximate r
discrepancy of but 1 ct per sq ft will sometimes amount to thou
the method is hardly accurate enough to estimate even the ap

Spinning-Mill. This mill has an attached picker-house, office se wings, and was built in Rhode Island in 1911. The following ls of construction: main mill, four stories; size, 263.17 by 131.67 ft; cker-house, 42.67 by 131.67 ft; one-story dye-house 55 by 85.67 ir-tank, and other towers; walls of hard brick; plank and wearing- nsverse I-beam framing, supported by cast-iron columns, except where both transverse and longitudinal framing is used; slag roof- on wooden transverse rafters; floors built for a live-load of 75 lb The cost of the buildings was $0.965 per sq ft.

tory Cotton-Mill. This mill is without basement. It was built, the fan-room and repair-shop additions in Georgia, in 1910. The the details of construction: mill, four stories; size, 272 by 128 ft; pair-shop, one story in height and 122.67 by 36 ft in plan; regular ction, that is, brick walls, hard-pine transverse floor-framing, wooden l plank floors, except for six bays of the fourth floor which have longitudinals in addition to the hard-pine transverse timbers, and ays of the roof-framing which have both longitudinal and transverse mbers, these having been found necessary in both cases because of of the alternate columns. These buildings have extensive monitors, ylights, stair-towers, etc. The floors are designed to carry a live per sq ft. The cost of the building was $0.715 per sq ft.

-Mill of Irregular Shape. This mill is considerably wider at n at the other and has a basement at one end. It was built in ts in 1911. The following are the details of its construction: mill, length, 311.67 ft and average width, 75.42 ft; five-story wing, .01 ft, with extensive pent-houses; stair and elevator-towers and rick walls, transverse wooden floor-framing, supported by cast-iron l brick walls; conditions at site demanded extensive foundations; fourth and fifth stories of one wall protected by wire-glass in metal floors built for a live load of 75 lb per sq ft. The cost of the buildings er sq ft.

tory Machine-Shop. This was built near Boston, Mass., in 1910. ilding is 200 by 136.375 ft with a connecting wing, 50 by 39.33 ft. walls; longitudinal, steel, I-beam framing; transverse, steel, saw- ht framing; plank roof covered with tar and gravel; 20-ft longitu- 5-ft transverse bays; steel I-beam columns; 4½-in cement floors ree bays which have a 1-in maple overflooring, a 1-in North Caro- termediate layer, a 3-in kyanized spruce-plank layer, and 4½ in of and extensive saw-tooth skylights. The cost of the buildings er sq ft.

ng for Manufacturing Automobiles. This building has forge- ons and was built in Connecticut in 1910. The main building

Mass., in 1909. It is 213 by 244.17 ft in plan, with extensive
and saw-tooth skylights. It has brick walls; longitudinal **I**-b
porting transverse **I**-beam girders in the first story, resting
transverse hard-pine girders supporting longitudinal girders f
skylight-framing; heavy, wooden floors and roof; wooden co
basement-floor; and foundations on concrete piles. The flo
to carry a live load of 100 lb per sq ft. The cost of the build
story basis, was $1.56 per sq ft.

A Two-and-One-half Story Picker-House. There is,
house and a one-story connecting passage between the two bui
above for the cotton mill of irregular shape, which were built i
in 1911. The picker-house is 64 by 95 ft in plan; the waste-h
the covered bridge 10 by 40 ft; and the average height of the l
The walls are of brick. The picker-house has transverse wooc
ported by wooden columns and has plank floors. The waste
transverse, steel **I**-beam framing and no columns, and con
The floors are designed to carry a live load of 75 lb per sq ft.
building, including plumbing, was $1.29 per sq ft.

**The Cost of Buildings of Mill-Construction in Philade
Vicinity.** The following five buildings were designed by St
Philadelphia, Pa., who submit data and descriptions, with the
FOOT. These buildings are within a very limited area, being i
miles of Philadelphia, and are of somewhat heavier constru
described above, the floor-loads varying from 120 to 150 lb p
cost ranging from $0.85 to $1.23 per sq ft. The average floor-l
the average cost $1.02 per sq ft. The two spinning-mills m
signed for average floor-loads of 120 lb and their average
per sq ft.

A Chocolate-Factory. This was built in Philadelphia, Pa.,
It has an ornamental exterior: walls of Saver and Fisher brick

er sq ft.

ng-Mill. This building was erected in Philadelphia, Pa., on ground
sy of access. Its exterior is a plain brick design. It is 69 by 269 ft
three stories in height, the story-heights throughout being 15 ft
op to top of floors. The floors throughout are calculated for a live
lb per sq ft. It has brick walls with concrete foundations; a slow-
or-construction with a finished flooring of maple; a slag roof; all
nd elevators in brick enclosures; and all openings in division walls
ith fire-doors. The cost of the building excluding the plumbing,
ctrical work, elevators, fire-protection and mechanical equipment,
and the cost of the building including the plumbing, heating, elec-
elevators and fire-protection, but excluding the mechanical equip-
1.34.

ing-Factory. This building was erected in Woodbine, N. J., on
and easy of access. Its exterior is of brick, without ornamentation.
179 ft in plan and three stories in height. The basement is 10 ft in
the other stories 12 ft in height from top to top of floors. The
lculated throughout for a live load of 120 lb per sq ft. It has walls
ow-burning floors with yellow-pine finished flooring; a slag roof; and
s and elevators in brick towers. The cost of the building, exclud-
electrical work, fire-protection and mechanical equipment, but in-
ght-elevators and plumbing, was $1.01 per sq ft.

st of Buildings of Mill-Construction in the Middle West.
ng six buildings were designed by F. G. Mueller, Hamilton, Ohio,
ts data and descriptions with the costs of buildings of heavier con-
The floor-loads vary from 200 to 300 lb and the cost from $0.62
r sq ft. The paper-mill at Taylorsville, Ill., is partly of concrete
n, and was built at a cost of $1.30 per sq ft. Exclusive of the last-
lding, the average floor-load is 230 lb and the average cost $0.805

An Addition to a Foundry. This one-story, brick, foundr
erected in Hamilton, Ohio, is 432 by 63 ft in plan, and has a
86 by 46 ft in plan, and a one-story cupola-house, 28½ by 26½
has a wooden floor in the wing only and dirt floors elsewhere. I
foundations; a composition roof on 2¼-in sheathing, supported
girders, 6 by 12-in beams and 6-in cast-iron columns; an elevator
house; and all doors of tin-clad construction. The cost of the
$0.836 per sq ft.

A Paper-Mill. This was built in Monroe, Mich., and is a
basement brick building, 185 by 87 ft in plan, with an end-wing
It has heavy beam and girder floor-construction, designed to car
of 300 lb per sq ft; concrete foundations and a basement-part,
It is designed for one paper-making machine and four beaters, has
roofing and one skylight over the boiler-room. The cost was $0.

A Paper-Mill. This is an irregular-shaped brick building erec
worth, La., and is 356 by 168 ft in plan. About one-third of it
and the remainder one story in height. It has a heavy wooden
and post-construction; a stone foundation on cypress-grillage
floors designed to carry heavy paper-making machinery with
250 lb per sq ft. The cost of the building was $0.96 per sq ft.

A Warehouse. This is a one-story-and-basement brick buildi
Hamilton, Ohio, and is 38 by 50 ft and designed for a live load
ft. It has a cement floor in the basement; 10 by 14-in girders, 8
and 10 by 10-in posts supporting 3⅝-in flooring; 10 by 14-in gir
round, wooden posts carrying 2¼-in sheathing and composition
cost of the building was $0.62 per sq ft.

A Paper-Mill. This was built in Taylorsville, Ill., and has a
two stories in height and 49 by 130 ft in plan; a one-story part,
in plan; and a one-story wing, 42 by 144 ft in plan. There is a b
almost the entire building. The foundations are of concrete
cement floors in the basement. The first floor is of reinforced
and slab-construction, designed for a live load of 250 lb per sq
floor of mill-construction, supported by cast-iron columns, 14 by
girders and 12 by 16-in wooden beams; and most of the roof is
steel trusses and wooden purlins. The second floor was designed
of 150 lb per sq ft. There are extensive skylights, pent-houses,
of the building was $1.30 per sq ft.

The Cost of Buildings of Mill-Construction in Toron
The building described in the following paragraph was designed
Rolph, of Toronto, Canada, who submit data of a warehouse-bui
floor-openings and windows and other outer wall-openings pr
approved manner, and erected at a cost of $1.12 per sq ft.

is of cut stone. The floors are designed to carry 300 lb per The cost of the building, exclusive of the heating, elevators, per sq ft.

ory-and-Basement Warehouse. This was built in Winnipeg, 62 ft 6 in by 86 ft 6 in in plan. Heavy fir timbers were used for as a 6-in fir-plank solid floor with ⅞-in maple overflooring; tors in brick towers; brick walls with the openings in the rear e building protected. The floors were designed to carry a live er sq ft, and the cost of the building, excluding the heating, etc., ft.

se. This is a six-story-and-basement building, erected in Saska-nd is 50 by 112 ft in plan. The floors are of 6-in fir, with ⅞-in ing, and are supported by heavy fir timbers. The building has a front of pressed brick and cut-stone trimmings; some of the otected by wire-glass windows; and the stairs and elevators are in The floors were built to carry 350 lb per sq ft, live load, and the xclusive of the heating, elevators, etc., $1.44 per sq ft.

y-and-Basement Warehouse. This was built in Edmonton, 50 by 137 ft in plan. The floors are of 6-in fir with ⅞-in maple The building has brick walls and the front and one side wall pressed bricks with stone trimmings. It has the openings in rotected and the stairs and elevators are in brick shafts. The enough for two additional stories and the floors are designed , live load, per sq ft. The cost of the building, exclusive of rs, etc. was $1.54 per sq ft.

f Buildings of Mill-Construction in Vancouver, Canada. escribed in the following paragraph was designed by Dalton & ouver, Canada, who give data of a warehouse with floors designed age load of 500 lb per sq ft and costing $1.09 per sq ft. Although mbers and the heaviest wall-hangers and beam-hangers were floors built of the maximum thickness, the cost was extremely doubt was partly due to the proximity of the timber and the nsporting it by water.

sq ft on the ground floor to 250 lb on the top floor, the average li
500 lb per sq ft. The walls and posts were designed to carry t
stories, with a live load of 225 lb per sq ft. The cost of the buildi
of the heating and office and warehouse-fixtures, was $1.09 per sq f

22. Cost* of Brick Mill-Buildings of Slow-Burning Co

Approximate Cost of Brick Mill-Buildings. Mr. C. T. Mai
series of diagrams showing the cost in New England, in 1910, PER
OF FLOOR SPACE, of BRICK MILL-BUILDINGS of different sizes, fro
stories in height, and of the type known as SLOW-BURNING. Th
are made for total floor-loads of about 75 lb per sq ft. The figur
the diagrams are given on the following page. The costs incl
foundations and plumbing, but no heating, sprinklers or lighting.

Modifications of the Costs given in Table I: (1) If the soil
conditions of the site are such as to require more than ordinary
the cost will be increased.

(2) If the building is to be used for ordinary storage-purposes wi
and no overflooring, the cost will be decreased from about 1
low buildings to 25% for small, high ones, about 20% being
allowance.

(3) If the building is to be used for manufacturing and is subst
of wood, the cost will be decreased from about 6% for large, one-s
to 33% for small, high buildings; 15% would usually be a fair all

(4) If the building is to be used for storage and built with low st
stantially of wood, the cost will be decreased from 13% for la
buildings to 50% for small, high buildings; 30% would usually b
ance.

(5) If the total floor-loads are more than 75 lb per sq f
increased.

(6) For office-buildings, the cost must be increased to cover
architectural treatment and the interior finish.

I. Cost of Brick Mill-Buildings per Square Foot of Floor-Area

ft	50	100	150	200	250	300	350	400	500
ft	One story								
	$1.90	$1.66	$1.58	$1.54	$1.51	$1.49	$1.48	$1.47	$1.46
	1.52	1.29	1.21	1.18	1.16	1.15	1.14	1.13	1.13
	1.41	1.21	1.12	1.08	1.06	1.04	1.03	1.02	1.02
	1.32	1.09	1.02	0.98	0.96	0.94	0.94	0.93	0.92
	Two stories								
	2.00	1.62	1.52	1.47	1.44	1.41	1.39	1.38	1.36
	1.50	1.21	1.13	1.09	1.06	1.05	1.04	1.03	1.02
	1.34	1.08	1.01	0.97	0.94	0.92	0.92	0.91	0.90
	1.22	0.97	0.90	0.86	0.84	0.82	0.81	0.80	0.86
	Three stories								
	1.98	1.57	1.47	1.42	1.39	1.38	1.36	1.35	1.34
	1.47	1.17	1.07	1.03	1.01	1.00	0.98	0.98	0.98
	1.30	1.05	0.98	0.94	0.91	0.89	0.88	0.87	0.86
	1.18	0.93	0.86	0.82	0.80	0.78	0.77	0.76	0.76
	Four stories								
	2.00	1.61	1.50	1.45	1.42	1.40	1.38	1.37	1.36
	1.38	1.17	1.10	1.05	1.02	1.00	1.00	0.99	0.98
	1.32	1.08	0.97	0.93	0.90	0.88	0.88	0.87	0.87
	1.20	0.93	0.85	0.81	0.78	0.77	0.76	0.75	0.74
	Six stories								
	2.10	1.72	1.57	1.51	1.48	1.46	1.44	1.43	1.42
	1.53	1.21	1.12	1.08	1.05	1.04	1.03	1.02	1.02
	1.35	1.08	0.98	0.94	0.92	0.90	0.89	0.88	0.86
	1.22	0.96	0.86	0.82	0.79	0.78	0.77	0.76	0.76

PER SQUARE FOOT of a building 100 ft wide is about midway between that wide and one 125 ft wide; and the cost of a five-story building about mid- the costs of a four-story and a six-story building.

al Data for estimating costs of foundation-walls and other walls the following table:

Cost of Walls in Brick Mill-Buildings of Slow-Burning Construction

Number of stories	1	2	3	4	5	6
ns, including excavations lin ft:						
le walls	$2.00	$2.90	$3.80	$4.70	$5.60	$6.50
walls	1.75	2.25	2.80	3.40	3.90	4.50
ls sq ft of surface:						
le walls	0.40	0.44	0.47	0.50	0.53	0.57
walls	0.40	0.40	0.40	0.43	0.45	0.47

tor-towers for buildings up to 300 ft long. In buildings over
height, three stairways and three elevator-towers for buildings ov

Plumbing Fixtures. In buildings of more than two stories
each fixture, including the piping and partitions. Allow for tw
each floor up to 5 000 sq ft of floor-space, and one fixture for e
5 000 sq ft, or fraction thereof, of floor-space.

CHAPTER XXIII

FIREPROOFING OF BUILDINGS

By
RUDOLPH P. MILLER
DENT OF BUILDINGS, BOROUGH OF MANHATTAN, NEW YORK CITY

1. Definitions, Areas, Heights and Costs

. The term FIRE-PROOF, while now quite well understood by
still used in a very broad sense by the public. To be strictly
building must be constructed and finished entirely with incom-
rials, and any of these materials, such as steel or iron, which are
fected by heat or streams of water must be efficiently protected
erials which are not so affected. This precludes the use of wood,
sed or not exposed, also all exposed steel or iron, common glass
lding stones. It is safe to say that there are very few buildings
ry that are absolutely FIRE-PROOF. There are many, however,
t be destroyed by fire, and in which the salvage would probably
m 60 to 80%; and it is the latter class which is generally meant
n FIRE-PROOF is used. Incombustible buildings, and buildings of
ruction protected to a greater or less degree from the flames, are
vertised as FIRE-PROOF; but such buildings should be considered
W-BURNING. It is undoubtedly the duty of every architect to be
l concerning the fire-proof qualities of all materials that enter
ruction and finishing of buildings, and to know how to use these
he best advantage. His choice and use of materials is then limited
character of the building and the interests of his clients. It is
rnish this information in a concise manner in this chapter.

Definitions. Municipal definitions as to what constitutes FIRE-
UCTION have a great bearing on the construction of buildings
jurisdiction, and those of the two largest cities are therefore

efinition.* "The term FIRE-PROOF CONSTRUCTION shall apply to
n which all parts that carry weights or resist strains,† and also all
and all interior walls and all interior partitions and all stairways
or enclosures are made entirely of incombustible material, and in
tallic structural members are protected against the effects of fire
of a material which shall be entirely incombustible, and a slow heat
d hereinafter termed FIRE-PROOF MATERIAL. Reinforced concrete
his ordinance shall be considered fire-proof construction.
rials which shall be considered as filling the conditions of fire-
g are: First, burned brick; second, tiles of burned clay; third,
ent concrete; fourth, terra-cotta."

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tter is left in its original form. The editor-in-chief is not responsible for
tuation, etc.
meant.

Table I. Limiting Heights for Non-Fire-proof Buildings

City	All buildings	Hotels	Schools	Hospitals and asylums	Residence-buildings
New York, N. Y.	75 ft	36 ft 6 in	36 ft 6 in	36 ft 6 in	75 ft
Chicago, Ill.	90 ft	Five stories and basement	Three stories and basement	Two stories	Five stories and basement
Philadelphia, Pa.	Six stories 85 ft	Four stories	Four stories	Two stories	Four stories
St. Louis, Mo.	75 ft	75 ft	Two stories above basement	Two stories above basement	....
Boston, Mass.	75 ft	....	....	....	65 ft
Cleveland, O.	60 ft 80 ft 100 ft	Ordinary construction Mill-construction Semifire-proof construction			
Baltimore, Md.	85 ft	45 ft	45 ft	45 ft	70 ft
Pittsburgh, Pa.	90 ft	....	....	....	....
Detroit, Mich.	100 ft	100 ft	More than 1 500 people	100 ft	100 ft
Buffalo, N. Y.	72 ft	....	....	....	....
San Francisco, Cal.	55 ft 84 ft	Ordinary construction Mill-construction			
Newark, N. J.	65 ft	50 ft	40 ft	40 ft	....
Washington, D. C.	60 ft	55 ft	....	55 ft	55 ft
Kansas City, Mo.	....	Four stories	Three stories	Three stories	Four stories
Seattle, Wash.	70 ft	....	....	Three stories	....
New Haven, Ct.	65 ft 80 ft. Mill-construction Four stories	55 ft Three stories	45 ft Three stories	45 ft Two stories	55 ft Four stories
Omaha, Neb.					

Definition. Buildings required to be FIRE-PROOF shall be
ith walls of brick, stone, Portland cement concrete, iron or steel,
beams or lintels shall not be placed, and in which the floors and
f materials provided for in Section 106 of this Code. The stairs
andings shall be built entirely of brick, stone, Portland cement
or steel. No woodwork or other inflammable material shall be
he partitions, furrings, or ceilings in any such fire-proof buildings
ever, that when the height of the building does not exceed twelve
e than 150 feet, the doors and windows and their frames, the
gs, the interior finish when filled solid at the back with fire-proof
he floor-boards and sleepers directly thereunder, may be of wood,
etween the sleepers shall be solidly filled with fire-proof materials
to the underside of the floor-boards.

eight of a fire-proof building exceeds twelve stories, or more than
oor surfaces shall be of stone cement, rock asphalt, tiling, or
stible material, or the sleepers and floors may be of wood treated
s approved by the Board of Buildings, to render the same fire-
side window frames and sash shall be of metal, or of wood cov-
l. The inside window-frames and sash, doors, trim, and other
nay be of wood covered with metal, or of wood treated by some
d by the Board of Buildings to render the same fire-proof."

f the Code refers to fire-proof floors. These may be constructed
ement-concrete, and, in fact, of any material that will success-
tests prescribed by the Code. Before any floor-construction
k or tile will be passed by the department, however, it must be
gth and fire-resistance under very rigid conditions, and the con-
stand the test successfully.*

proof Construction Should be Employed. A building should
ilt, and finished to conform to the purpose for which it is to be
ng containing but little inflammable material, and that not of
d not be as thoroughly fire-proof as one designed for the storage
ds, or for the protection of life in case of fire. The height of a
mportant factor in determining whether it should be fire-proof
te of increase in the difficulty of coping with fire in a building
hat of the increase in the height. The area covered by a build-
ortant, although in most instances interior division-walls may
ch practically cut up a building into a series of smaller buildings.
itations placed upon non-fire-proof buildings by various munici-
found in the following classification and in Table I, page 812.

eas for Non-Fire-proof Buildings

, 8 000 sq ft on an interior lot.
12 500 sq ft on a corner.
22 000 sq ft when facing three streets.
9 000 sq ft if of ordinary joisted construction.
12 000 sq ft if of slow-burning construction.
7 500 sq ft.
5 000 sq ft.

, Mill-Construction:
20 000 sq ft when facing streets on four sides.
15 000 sq ft when facing streets on three sides.
12 000 sq ft when facing streets on two sides.
9 000 sq ft when facing streets on one side.
5 000 sq ft on any lot when of hazardous occupancy.

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* *See page 827.*

concrete construction costs from 10 to 15% more per square foot o
than mill-construction and about 25% less than steel-frame an
fire-proof construction.† Figures given by J. P. H. Perry indic
forced-concrete construction adds from 2 to 20% to the cost of
tion for commercial buildings, with an average of 6.7% for var
and all classes of buildings in the United States. The increase in
tural-steel fire-proof construction over reinforced-concrete cons
aged 6.4% for fourteen buildings of all classes in various localities.

Divisions of the Subject. In constructing fire-proof buildin
sary to consider:

(1) Materials to be used.
(2) Form of construction.
(3) Protecting devices.
(4) Extinguishing appliances.

This general order is followed in the discussion of the subject i

2. Fire-Resistance of Materials

Effect of Heat on Building Materials. All materials of co
more or less injuriously affected by high temperatures. Furthe
COMBUSTIBLE material is not necessarily FIRE-RESISTING, as, for i
The value of various materials in fire-proof construction is inc
following paragraphs.

Brickwork. Common brickwork, when of a good quality, will s
to fire for a considerable length of time, but in a severe conflagrati
side of a wall expands, often to the point of throwing the wall,
crack, shell, and are sometimes melted. Experience has shown th
are less affected by heat than thin walls, and that hard-burned
better than soft or under-burned bricks. In buildings which a
large quantities of inflammable material it is undoubtedly bett
walls with porous furring-tile or hollow brick. In the Baltim
Francisco fires, it was demonstrated that for outside walls brick
a fire-proof material to any other material used in wall-constructi

and Marble are usually ruined if not totally destroyed by an
They are the least desirable of all stones for use in a fire-proof
the granites come next.

when fine-grained and compact sometimes stands fire without
, but in the case of a severe conflagration it is generally so badly
t has to be replaced.

a is made from clay by mixing it with water into a plastic mass,
ıme into the form desired and baking it at a high temperature in
e usual structural form the shaping is generally done by forcing
ass through a special die by means of machinery. Ornamental
ıst generally be shaped by hand.

l Terra-Cotta. This material, and especially that which has a
, is well adapted for the trimmings of a building that is intended
f. It should, however, be made heavy enough to carry both its
d its share of the wall-load.*

Terra-Cotta. Terra-cotta, as used for floor-arches, column and
on, and for building light, hollow walls, is made of three differ-
ons, the material being known as DENSE, POROUS, and SEMI-
ling to the method of manufacture.

ng is made from a variety of clays. Some manufacturers use
re-clay, and combine it with potter's clay, plastic clays, or tough
is very dense and possesses high crushing strength. In outer
o the weather and required to be light, it is very desirable. Some
furnish it with a semiglazed surface for the outer walls of build-
h use it has great durability, and effectually stops moisture. In
ling for fire-proof filling, care should be taken that the tiles are
s, sound and hard-burned.

l Semiporous Terra-Cotta is made by mixing sawdust with the
ust being destroyed by the action of the heat, leaving the material
us. A small proportion of fire-clay mixed with the plastic clay
ut not essential. The proportion of sawdust should be from
cording to the toughness of the clay used. Care is required in
manufacture to have the work of mixing, drying and burning
ne. The burning should be done in down-draught kilns by a
The product should be compact, tough and hard, and should
ck with metal. Poorly-mixed, pressed, or burned tiles, or tiles
sandy clays, present a ragged, soft, and crumbly appearance, and
ble. When properly made, porous terra-cotta will not crack or
equal heating, or from being suddenly cooled with water when in
tion. It can be cut with a saw or edge-tools, and nails or screws
driven into it to secure interior finish, slates, tiles, etc. As a
-resistant and non-conductor for the protection of other materials,
ked very high.

s Tiling. This material was introduced by those factories which
lay in the manufacture of tile, to enable them to compete with
orous material. During the process of grinding the clay, about
d coal is mixed with it. This coal aids in the burning of the
lso makes it lighter and more or less porous. Tiling made by
admitted to be a much better fire-resistant than the solid or
. E. V. Johnson says: "personally, I believe that good semi-

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* "Fire Prevention and Fire *Protection,*" *J. K. Freitag.*

porous fire-clay tile is fully as efficient as a fire-resisting material as the st makes of porous terra-cotta."

Strength of Terra-Cotta. (See, also, page 276.) In tests made at C University for the building authorities of New York City on terra-cott taken from material delivered in the open market, the following c STRENGTH was developed:

Table II. Crushing Strength of Terra-Cotta

Description of material	Position of cells in test	Compressive st lb per sq i	
		Gross area	N
Dense tile...........................	Vertical Horizontal	1 864 585	
Semiporous tile.......................	Vertical Horizontal	1 027 257	

The inequality in strength of the two materials can be overcome thicker webs and shells for the semiporous or porous material. In th of WEIGHT, porous and semiporous terra-cotta have the advantage ov tile. Dense tiling, when heated and cooled by water, is liable to cr the sudden contraction; "blocks with two or more air-spaces are ve to have the outer webs destroyed under this action. Even if not co water, other fires have shown that hard-burned terra-cotta will crack to pieces under severe heat alone." * The experience of the recent c tions in Baltimore and San Francisco fully bears out this stateme collapse of the floors of one of the buildings in Baltimore was largel the weakening of the terra-cotta arches by reason of the breaking o outer shells. Porous terra-cotta is non-heat-conducting in itself, and qualities will usually resist fire and water successfully; but if the pro not burned at a sufficiently high temperature to consume all of the saw throwing of cold water upon the heated surfaces will cause an exp disintegration due to the absorption of the water and its conversion into Porous terra-cotta absorbs water freely, and if allowed to freeze whe more or less injured. If the process is permitted to continue, the block so weakened that they are unsafe for use.

Concrete Blocks and Concrete Tiles. Numerous forms of blocks and tiles are manufactured of Portland-cement mortar or co use as substitutes for brick, stone and terra-cotta. Concrete blocks are the DRY PROCESS by tamping a dry-concrete mix into shape in forms, WET PROCESS which consists of pouring a semiliquid or SLUSH-MIX into m curing the product by air or steam. Various types of machines have been for the manufacture of the blocks by the former method, a typical being that of the Ideal Concrete Machinery Company of South Be Concrete hollow tile is being made for the same uses as terra-cotta partitions and floors in general, and for enclosure-walls as well as for in residences. For wall-bearing purposes, the tiles are usually filled a layer or two where the beams rest upon them. In hollow-block con distinction should always be made between the strength of the blo

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[*] "Fire Prevention and Fire Protection," J. K. Freitag.

the core-holes vertical and when laid with the core-holes horizontal, ength, in the latter position, approximates only one-half of what it is rmer. The specifications given in the Proc. Nat. Asso. of Cement 11, are generally accepted as the best practice in the manufacture of blocks. (See, also, Chapter III, page 233.)

ete Tile. Concrete building tiles have been used extensively for s in Chicago, Ill., Rochester, N. Y. and the suburbs of New York he shape and size of the blocks vary with the make of the product. a WET-PROCESS tile made by the Concrete Products Company, of k City, is in extensive use in various parts of the country, for enclos- interior partitions and combination concrete-and-tile floors of resi- The COMPRESSIVE STRENGTH in pounds per square inch of Tilecrete the Bureau of Buildings, New York City, in 1911 was as follows:

Table III. Compressive Strength of Tilecrete.*

Dimensions and use			Cells vertical		Cells horizontal	
ss.	Kind of tile	Number of cells	Gross area, lb per sq in	Net area, lb per sq in	Gross area, lb per sq in	Net area, lb per sq in
	Wall-tile	2	....	....	320	746
	Wall-tile	2	528	1 510	351	1 228
	Corner-tile	2	633	1 580	....	....
	Wall-tile	4	510	1 050	360	1 066

her information may be obtained from the catalogues of the Concrete Company, New York City; Chicago Structural Tile Company, Chicago, more, Rauber and Vicinus, Rochester, N. Y.; Concrete Stone and Sand , Youngstown, Ohio.

ete. Stone concrete, under the action of heat, is affected much the y as brickwork. The heated surface expands, and as the concrete is oor conductor, the other surface remains cool and either cracks or rping. The heat also affects the strength and texture of the concrete, disintegration of the concrete to a depth of about 1 in. Often the palls off with a report. If water is applied after the heat, the surface l away to the depth of the affected part. These effects vary somewhat stone used in the aggregate. Gravel and granite, on account of the between their coefficient of expansion and that of the concrete, are spall. Limestone calcines under the action of heat and is especially destruction for some depth by the water. Trap-rock is the most satis- naterial to use, from the standpoint of fire-resistance as well as that th. If there is no application of water after the fire and the surface d to cool off gradually, the concrete may set again and become hard. well, however, to rely on this. (See, also, Chapter III, page 245, for of heat on concrete fireproofing.)

oncrete. Blast-furnace slag has been used as the aggregate in con- h very satisfactory results as to both fire-resistance and strength.*

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series of tests and description of materials, see pamphlet issued by the Carnegie pany, 1911, "Furnace *Slags in Concrete.*"

be determined from load-tests and a high FACTOR OF SAFETY sh
The practice in New York City is to take one-tenth of the BREA
the WORKING LOAD.

Corrosive Action of Cinders. When cinder concrete is u
steel, either as a protective covering or as a part of a concrete co
corrosive effect of cinders must be guarded against. A discussio
ject will be found in Chapter XXIV, pages 961 and 962.

Mortars, Plasters and Plaster of Paris. Mortar and plast
sarily enter into the composition of all masonry buildings, wh
brick, stone, or terra-cotta. That ordinary lime mortar, when w
endure for unlimited periods of time, in dry situations, has be
actual use. Hydraulic-cement mortars are equally durable in
places. For laying brickwork or tilework in first-class buildings
sand mortar is preferable to any other; and cement mixed wit
gives greater strength than lime and sand alone. Regarding
qualities of mortars and plaster compositions there has been muc
the truth of the matter seems to be that all such compositions
the action of heat up to a certain degree, when they are affect
or another, depending not only upon the composition but in large
their body, and upon the way in which they are used. Lime mo
was formerly considered as the most satisfactory, so far as fir
concerned; but since the improvements in cement-manufacture,
is generally preferred. Lime plaster, applied on wire lath, will wi
degree of heat without injury, but is liable to be washed away
streams of water. Hard wall-plasters, or patent plasters, whe
brickwork or metal lath, are in all cases equal in heat-resistan
lime, and many of the patent plasters will stand the combined
and water longer than the common mortars.

Plaster of Paris. Compositions of plaster of Paris and broke
chips, or sawdust are non-conductors of heat and possess fire-resis
of considerable importance; and on account of their lightness a
are often used in fire-proof or semifire-proof buildings. In Fra

of steam-packings, corrugated sheathings, roof-coatings and shingles, , and building-lumber, insulating sheathing and blocks, asbestos tains, various forms of preservative and fire-resisting compounds, utes for wall-plaster and stucco. The value of these products lies r heat-conductivity and incombustibility.

s Building-Lumber is made in standard sheets, 36 by 48, 42 by 48 96 in in size, and varying in thickness from ⅛ in (about 1 to 1¾ lb weight) to 2 in (from 16 to 20 lb per sq ft in weight). When seasoned than ordinary wood, takes nails and screws, and it can be manipulated tools and machinery such as are used for working iron; it is too hard r wood-working tools. It is sufficiently elastic to withstand ordinary xpansion and contraction of surrounding parts, wind-pressure and l in large pieces, it can be bent around slight curves without

s Corrugated Sheathing is corrugated asbestos building-lumber, vith sheet steel of from No. 24 to No. 27 United States gauge or with netting. It is applied in the same way that corrugated iron is her nailed to wooden strips bolted to the purlins, or clipped directly ns by clips of hoop-iron or wire. It comes in standard sheets, 27½ in lengths of 4, 5, 6, 7, 8 and 10 ft.

s Roofing-Shingles, suitable for wooden-roof construction, possess g qualities far superior to wooden shingles. The advantages claimed e-proof qualities, toughness, elasticity and lightness in weight; ease ation, cutting, sawing and shaping to fit dormer windows, chimneys, their immunity from the corrosive action of salt air. The principal manufacturing asbestos building-products are the Johns-Manville New York City, the Keasbey & Mattison Company, Ambler, Pa. bestos Manufacturing Company, Lachine, Canada.

s-Protected Metal consists of steel sheets of from No. 28 to No. 20 es gauge, coated on both sides with asphaltum-compounds contain-natural oils, and covered with layers of asbestos-felt put together

ures there were many instances of failure in steel columns due
insufficient protection.

Cast Iron. "As the result of tests and actual experience in
it may be stated that unprotected cast iron can stand practicall
to temperatures of 1 300 or 1 500° F. while carrying very heavy l
frequent applications of cold water while the metal is at a red h
tests at the Continental Iron Works, referred to in the precedin
temperature of nearly 1 300° F. was reached before the cast-iron co
fail. The contents of most mercantile buildings, when burnin
probably generate a heat exceeding at times 2 000° F. Consequ
columns, when unprotected, are almost sure to fail in such a fire
ing or breaking. No building in which unprotected iron or ste
used can be considered fire-proof; but in many classes of buildin
cast-iron columns might safely withstand any heat to which they
be exposed. From a fire-resisting point of view, when there is
covering, cast-iron columns are unquestionably preferable to stee

Fire-proof Wood. To meet the requirements of certain pr
New York City Building Code, an attempt has been made to pro
wood. The processes for rendering wood fire-proof, in general,
pregnating its fibers with certain chemicals. After the firep
the lumber should be thoroughly kiln-dried before it is used.
are more easily thoroughly treated than the hardwoods, the
being particularly difficult to handle.

"The treatment of the wood to render it fire-proof slightly rais
point of the wood. The treated wood is harder to light than
wood, taking two to three times as long to ignite. The amount of
when exposed to the action of a flame is from 5 to 12 per cent
case of an untreated wood than in the case of a treated wood.
wood furnished more flame than the treated wood. The untre
sustain flame longer than the treated wood after the source of

nty seconds after the removal of the test-piece from the furnace, and ied and uncharred section at the center of the specimen should be an 50 to 70 per cent of the original cross-section, depending on the wood under test." If the wood has been thoroughly treated, a it after having been exposed to flame and withdrawn, will show flame. Other tests have been suggested and used but need not be here. At the present time fire-proof treatment of wood is being by The Standard Wood Treating Company and the Electric Fire-ompany, both of New York City.

ass. The introduction of this material has made it possible to protection in many cases, without the necessity of disfigurement due ters. Wire-glass is either RIBBED, ROUGH, MAZE, COBWEB or POLISHED wire embedded in its center during the process of manufacture.

mperature at which the wire is embedded in the glass insures ad- veen the metallic netting and the glass, and the two materials become eparable, so that if the glass is broken by shock, by intense heat, or cause, it remains intact." It is this property of remaining intact it its fire-retarding qualities. Although fire and water may cause pread throughout the glass the wire holds the pieces so firmly that not pass through it. Many severe tests during actual fires have demonstrated the truth of the above claim. For warehouses and ie RIBBED or MAZE glass is generally preferable; but for offices, or lear transparent glass is desired, the POLISHED PLATE is nearly if s acceptable as the same glass without the wire, the effect being the at obtained by looking through a window with a screen on the out- re FIRE-RESISTANCE is the desired feature, the following requirements satisfied. The thickness of the plate at the thinnest part should than ¼ in, and the plane of the wire mesh should be midway between rfaces of the glass. No wire should be smaller than No. 24 Brown gauge. The unsupported surface of the glass should not exceed fire. The Bureau of Buildings of New York City makes the fo tions regarding fire-proof paint:

"(1) The term FIRE-PROOF PAINT shall be understood to mea tion used to cover the surfaces of wood or other materials for protecting the same against ignition.

"(2) No fire-proof paint will be considered satisfactory unles the wood or other material to which it is applied that the same or glow after having been subjected to the flame of a gasoline minutes.

"(3) Before applying fire-proof paint to any material the su cleaned.

"(4) Application of fire-proof paint must be repeated whene that the material to which it is applied is no longer protected t cation No. 2."*

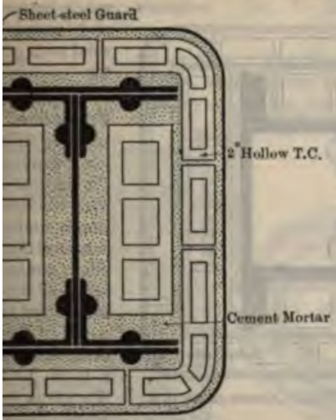
3. Column-Protection

Girder and Column-Protection. As the columns and girde form the BACK-BONE of the structure, it is of vital importance th thoroughly protected from heat. As a rule, the manner of p structural elements depends quite largely upon the floor-sy Where concrete is used for the floor-construction it is generally for incasing the columns and girders; where hollow tile is use the same material is almost invariably employed for protecting t The methods used for protecting girders are described in Subdi 863 to 866, of this chapter. (See, also, pages 780 to 782.)

Necessity for Column-Protection. It is now generally r iron and steel columns should be incased with some material oughly protect the metal against fire. In 1896 a committee of Society of Mechanical Engineers, in conjunction with representat

Table IV. Tests of Protective Coverings

Materials under test	Temp. on face of protective material, degrees Fahr.	Temperature of plate at back of protective material, degrees Fahr.		
		Before heating	After heating for 2 hr	Heat-transmission
ta: dense, hollow, 2 in thick..	1 700	75	223	148
ta: semiporous, solid, 2 in	1 700	73	244	171
f Paris and shavings, 2 in	1 700	69	159	90
Paris and asbestos, 2 in thick.	1 700	70	163	93
f Paris, wood fibers, and in- earth, 2 in thick.............	1 700	72	167	95
of ground cinders, 1$\frac{5}{16}$ in	1 700	73	363	290
oncrete, on metal lath, 2 in	1 700	66	248	182
h and patent plaster, about hick over 1 in air-space........	1 700	76	296	218



in cement mortar, occasionally, bound with copper wire at inter-

Hollow-tile Protection. Plate-and-angle Column

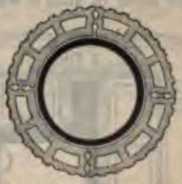


Fig. 2. Hollow-tile Protection. Cylindrical Column

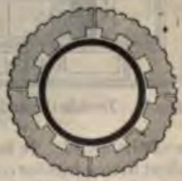


Fig. 3. Ribbed-tile Protection. Cylindrical Column

Cotta Column-Protection. *Fig. 1 shows the manner in which built- are protected in the best class of fire-proof buildings when tile fire- sed. Figs. 2, 3 and 4 show common methods of protecting cylin-*

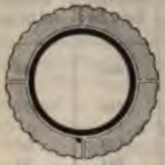


Fig. 4. Solid-tile Protection. Cylindrical Column

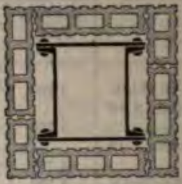


Fig. 5. Hollow-tile Protection. Built-up Box Column

Fig. 6. Hollow-
tection. Squar
section

drical columns, and Figs. 5 and 6 columns of rectangular cross-sectio
steel guard, shown in Fig. 1, is often employed in mercantile and manufa
buildings. and put on to a height of 4 or 5 ft above the floor. The effici
this construction is greatly increased by wrapping the columns with w
before plastering, although it is not a common practice. To insure the
tion of the metal under the most trying conditions, it is imperative t
protective covering shall not be detached by the streams from the
hose, and thus expose the steel. This can be positively guarded agai
by using two layers of tiling or concrete and wrapping the inner la
metal lathing. Fig. 7 shows a column protected in this way, the con
being essentially that adopted in the Fair Building in Chicago, I
inner layer of tiles is wrapped with wire lath embedded in the mortar
spaces betwee
and metal
solid with cem
tar.

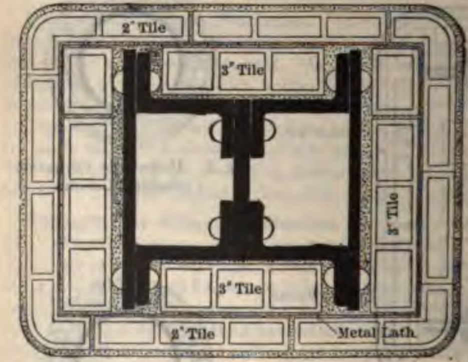
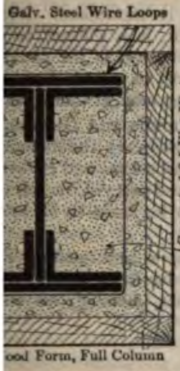


Fig. 7. Double-tile and Metal-lath Column-protection

**Concrete
Protection.**
concrete is t
for column-p
the way to c
most efficient
tion is undou
surround the
cinder concre
inside of a p
set around th
a coat of liqu
being first ap
a brush to t
The plank fo
be set at leas
side of the metal. It is generally conceded that this forms one of
efficient fire-casings for columns, and, in addition, lends added stiffness t
members embedded in it. It is advisable to reinforce the concrete or an
means of metal lath to the steel column. There are two general m
use in applying the concrete. Fig. 8 illustrates a column which is first
spirally with No. 10 gauge galvanized wire, 12 in on centers, to aff
for the concrete. The wood forms are placed the full length of th
and the concrete poured from a hole in the ceiling above. A sh

or stone concrete of 1 : 2 : 5 mix may be used. Fig. 9 shows a form
ards, made in sections from 4 to 6 ft in length and provided with
h end. The concrete may be thoroughly tamped about the column



crete Column-protection
Wooden Form

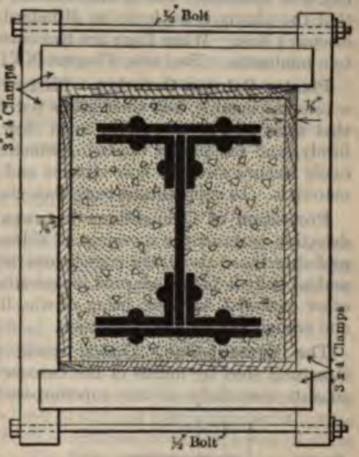
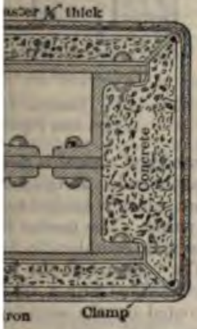


Fig. 9. Concrete Column-protection and Wooden Form

tion is placed and filled. Fig. 10 shows a method of furring the
stiffened wire lath, which serves as a substitute for the wooden
at the same time anchors the concrete to the steel. A similar



crete Column-protection.
-lath Furring

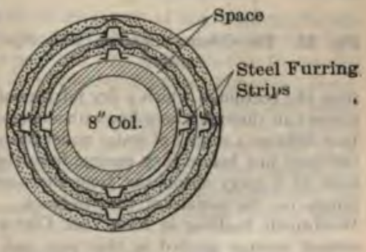


Fig. 11. Metal-lath and Plaster Column-protection

be employed *to obtain an air-space by placing immediately*
olumn an envelope of metal lath with a 2-in layer of concrete.
ings with reinforced concrete floors, the columns are protected

and the plaster tends to promote CORROSION in the m
easily washed away by hose-streams and subject to greater dan
materials. In unimportant work their cheapness may, at times,

Protection of Connections between Columns and Girde defective parts of the coverings of columns, whatever the mat probably those about the connections with the beams and gir undoubtedly is better adapted for covering these parts of the c other material, because, being elastic, it can be made to fit and around any form of connection.

The Cement-Gun. During recent years, a new method structural steel by means of the CEMENT-GUN has been introdu consists essentially of two superimposed tanks, forming two

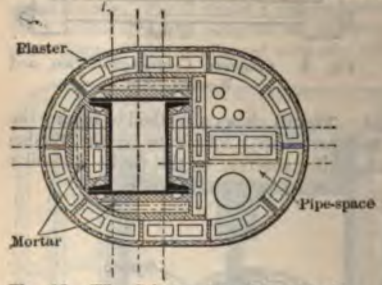


Fig. 12. Tile Column-protection with Pipe-space

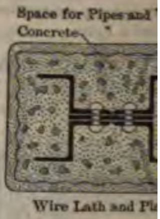


Fig. 13. Concrete
tion with Pi

from the bottom of which a dry mixture of sand and cement is
pressed air through a hose-line with a nozzle at the end. To this

4. Fire-proof Floor-Construction

oof Floors. In the study of fireproofing-materials by far the great- on has been given to FLOOR-CONSTRUCTION; and of the very large types which have been developed, the characteristic and leading ere considered.

ments for a Fire-proof Floor. It goes without saying that a fire- must be made of incombustible materials. It seems unnecessary, ention that it must resist as much as possible the transmission of to afford thorough protection to the metal incased by it or forming l part of it. The materials used should not disintegrate or otherwise xposed to heat or flame. They should also resist the action of water be used to extinguish a fire. The floor-construction should be essen- r-tight, so as to prevent damage by water in stories below. It should d to safely carry its load at all times. The New York City Building describing certain acceptable forms of fire-proof floors, provides for on other types as a precedent condition for their approval. More ty tests have been made under the auspices of the New York City and these, together with a few made by the authorities of other cities, ractically all that have been made in this country. The British ntion Committee of London has also made a number of such

sts for Floors. The STANDARD FIRE-TEST of the American Society g Materials‡ is essentially the same as that required by the New Building Code and as the one used by the British Fire Prevention . Briefly, the test consists in subjecting the floor in question, carry- of 150 lb per sq ft, to a fire maintained at 1 700° F. for four hours; plying a stream of water, at 60-lb nozzle-pressure, for ten minutes; , after cooling, in increasing the load on the floor to 600 lb per sq ft.

b. Flat reinforced floors,
c. Sectional systems,
(4) Composition systems.

Brick Floor-Arches. The first attempt at fire-proof flo between wrought-iron beams was made by using BRICK ARCHES s the beams and resting on the bottom flanges, as illustrated by F

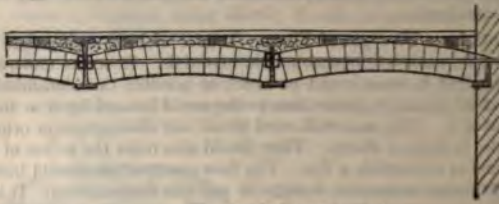
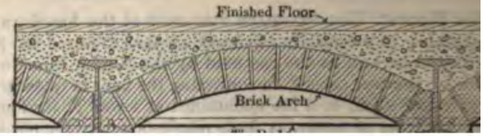


Fig. 14. Brick Floor-arch

this form of construction is used the bricks should be har bricks or hollow bricks of good shape, laid to a line on centers w with their lower edges touching; and all the joints should be cement grout. The bricks of one line should break joints with th adjoining, and in case there is more than one row, the joints of



perhaps, also, the most expensive. Its weight necessitates a heavier than is required for other types; and, on account of its appearance, d only to buildings of the warehouse type.

otta or Tile Floor-Arches. TERRA-COTTA or TILE as a fire-proof d the relative merit of dense, porous and semiporous tile have been n page 815. For floor-construction the semiporous tile is probably it is a compromise between the advantages and disadvantages of the porous tile, particularly as to strength and fire-resistance. As indi- ge 828, six different types of terra-cotta floor-construction, including imber of systems, will be discussed. For these a great variety of sizes of blocks, of the dense, porous and semiporous material, are ed in this country. The largest company devoted to the manu- l erection of hollow-tile fireproofing-material is the National Fire mpany, New York and Chicago. Other large companies are Henry Son, New York; the Haydenville Company, Haydenville, Ohio; re Fire-proofing Company, Delaware, Ohio; and the Illinois Terra- ber Company, Chicago. Any one of these companies can make f blocks desired, except such as are covered by letters-patent, and, hey can make them in dense, porous and semiporous material.

ges of Tile Floor-Arches. Many architects prefer the use of A ARCHES in buildings because the setting of them causes less dis- the mechanics of other branches of the construction. During the CONCRETE ARCHES the continual dripping of water and bits of con- eres seriously with other work. The work of installing tile arches more rapid than for other types and it is not necessary to wait for y out. The quality of terra-cotta can be readily judged from its not only before it is put in place but also after it is set. Thus it quire the constant supervision necessary for materials that are mixed put in place.

ntages of Tile Floor-Arches. The principal DISADVANTAGE OF

necessary to use considerable concrete filling over the arch the w
floor-construction will usually greatly exceed that of the concrete s
this additional weight means, also, additional expense. The flo
liable to breakage and chipped blocks in the floor are not unas
perhaps more apt to occur when, as in some localities, the arche
bricklayers. The manufacturers claim that when they use the
better work can be expected.

Inspection of Floor-Arches. Flat arches of hollow tile require
TION during erection to see that broken or imperfect tiles are no
the ribs in END-CONSTRUCTION tiles abut opposite each other; t
are properly mortared and that all of the steelwork is properly prot
poor workmanship has been allowed to pass in order to avoid de
because it cannot be discovered until the centering is removed.
generally looks better on the top surface than it does on the bott

Setting of Tile Floor-Arches. Tile arches are always s
CENTERS suspended by bolts hooked over the tops of the **I** beams.
of 5 ft and over, the centers should be slightly CAMBERED. Bef
arches are set, all girders projecting below floor-beams should
covered on the bottom and sides, independently of the floor-con
protect the steel from rust it should have a good coat of Portland-
before the tiles are applied. After the centers are in place the be
be placed under the bottom of the beams and mortar slushed on t
entire sides of the SKEW-BACKS which rest against the floor-bea
be covered with just enough mortar to give them a perfect beari
up against the beams. After this, the INTERMEDIATE BLOCKS,
on one end and one side covered with a full bed of mortar, sh
into place. The KEYS should have mortar on both sides and o
METHOD KEYS are used and they should fit snugly, but not tight.
ditions should a key be rammed in place. It is better to use a sm
out the space left with either a solid slab of tile, or, if the openi
with a piece of slate." † "In setting tile arches it is very comm
arches in STRING-COURSES, first fitting all the skews, then all the
and finally all the keys. This is bad practice, as it loads the cent
and stringers, to excess, causing too great a deflection. In the
TION the arches should be built one by one, each being comp
next is started. In SIDE-CONSTRUCTION, where joints are br
nally, the arches should be keyed up or completed at the first
intermediates meet the lines of the key, thus completing the s
as rapidly as possible." ‡ All JOINTS in the arches should be fill
especially at the top.

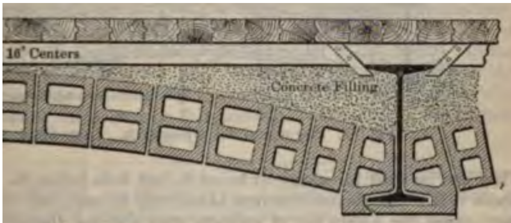


Fig. 16. Segmental Tile Floor-arch

;-strips are laid, all piping and wiring which must go above
e arches should be put in place. After the nailing-strips are
of the steel beams should be covered with a thin coat of Port-
sand grout, applied with a brush. The spaces between the
uld be filled with a 1 : 8 or 1 : 10 cinder concrete, finished
the tops of the strips. Some architects claim better results
tangular section, with nails driven horizontally into the ver-
the grip in the concrete. This method avoids the loosening
looring from any shrinkage of the strips.

ocks. In cases where the tops of the tile arches are 2 in or
ops of the steel beams, hollow tile blocks are sometimes used
op of the beams, as in Fig. 26. These blocks are lighter than
they do not strengthen the arches.

s. If the floors are to be finished with cement, the cement and
at least 2½ in and preferably 3 in thick above the steel beams,
cked out in sections of not over 6 ft square, with joints extend-
oncrete. When practicable the joints in one direction should

Segmental Tile Floor-Arches. "This form of arch is th cheapest. It is particularly adapted to warehouses, lofts, fact or wherever great strength is required and a flat ceiling is not n a light, strong arch is required in deep beams and a flat ceiling i this result can be obtained by using a metal-lath ceiling susp beams."* These arches are usually formed by either 6 or 8 set on the SIDE-CONSTRUCTION principle and bonded endwise li

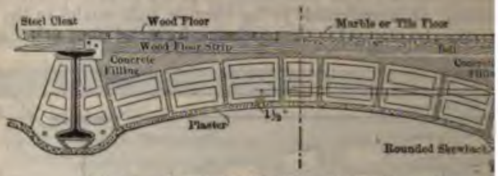


Fig. 17. Segmental Tile Floor-arch. Deep Skew

They can be used for spans up to 20 ft, but it is better to li about 16 ft. "END-CONSTRUCTION blocks may be used, but the tory, unless the arches are of uniform span and rise through the SIDE-CONSTRUCTION arch can be varied by increasing the upper or lower part of the mortar joint, but this cannot be do CONSTRUCTION method."*

Figs. 17 and 18 show typical forms of SEGMENTAL ARCHES. the arch tiles will run about 26 lb per sq ft for 6-in tile an

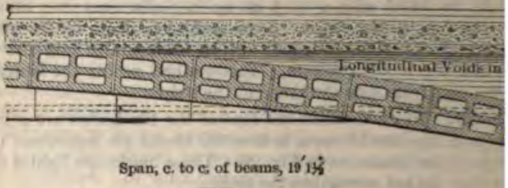


Fig. 18. Segmental Tile Floor-arch. Deep Beam. Dropp

tile. To these weights should be added the weight of concrete plaster, etc.

Thickness of Webs. "For general use the WEBS of segme ½ in thick for semiporous tile and ¾ in for porous tile. The SK be at least ¾ in thick for the first-named material and 1 in for t printing-establishments or any other building where a large a tion occurs the webs of all tiles must be designed in proportion the load they are required to carry." † These thicknesses a practice more particularly, where a stronger tile is produced than New York City webs are generally ⅝ in thick for semiporous an tiles.

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* Bevier, National Fire Proofing Company, New York

† E. A. Hoeppner.

The thrust of segmental arches is very considerable, so that it
provide tie-rods between the beams. A formula for determin- in the tie-rods and their diameter is given on page 871. To be the tie-rods should be placed at the center of the skew. Placing this manner, however, may cause them to project below the arch, giving an unsightly appearance to the ceiling. It is also to protect them when in this position. Occasionally the tie-rods h special tiles, as in Fig. 19.

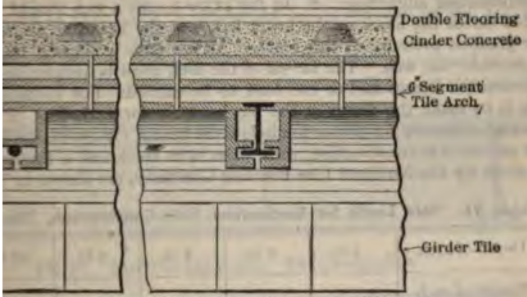

Fig. 19. Segmental Tile Floor-arches. Incased Tie-rods

Segmental Semiporous-Tile Floor-Arches. The safe loads t on 6 and 8-in segmental arches, with side-construction, semi- rise of one-eighth the span, webs and shells ⅝ in thick, and with ty of 7, as obtained from the tables of the National Fire Proofing as follows:

. Safe Loads for Segmental Semiporous-Tile Floor-Arches

Side-Construction Tile Floor-Arches. By this term is underst flat-tile arches in which the voids in the blocks run parallel with the be shown in Fig. 20. One advantage of this arch over the end-constru the BREAKING OF JOINTS that is effected in the setting of the blocks, by of which the failure of a single block does not impair the strength of beyond that block. The WEBS should not be less than ⅝ in thick. JOINTS are sometimes specified but should be avoided, as they incur se pense in manufacture and endless confusion and delay in setting, with

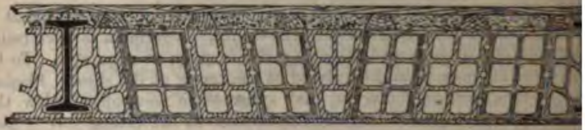

Fig. 20. Flat Tile Floor-arch. Side-construction

compensating advantage." * In the SKEW-BACKS a web should alway vided across the block at the lower flange of the beam, as at this poi the greatest pressure in this block. Arches have collapsed because to provide this web. The DEPTH of the arch must be proportioned to between the beams and to the load to be carried. For ordinary loa rule is to make the depth of the block 1¼ in for each foot of span, amount necessary for protection below the beams. SAFE LOADS for se tile arches, side-construction, with webs ⅝ in thick and a factor of sa as given by the National Fire Proofing Company, are shown in Table

Table VI. Safe Loads for Semiporous, Side-Construction, Tile Floor-

Depth of arch	6 in	7 in	8 in	9 in	10 in
Weight of arch per sq ft	24 lb	26 lb	27 lb	29 lb	34 lb
Span of arch, ft in	Strength of arch in pounds per square foot				
4 0	197	230	263	296	438
4 6	156	182	208	233	346
5 0	....	148	168	189	281
5 6	....	....	139	156	232
6 0	....	....	....	131	195
6 6	....	....	....	....	166
7 0	....	....	....	....	....

These loads represent the GROSS LOADS; so that for the SAFE LIVE LOADS of the construction, including the arch-blocks, fill, flooring, plastering, etc deducted. For blocks with thicker webs the loads may be increased propo Where no loads are given in the table, the spans are considered excessi depth of block specified. The weights of arch given in the table are for t blocks. If thicker webs are used, the weight of block must be taken propo greater.

End-Construction Flat Floor-Arches. In this construction the voids of the individual blocks run at right-angles to the beams, so tha

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* Bevier, National Fire Proofing Company, New York Ci

blocks is endwise of the tile. It has been conclusively demonstrated
tiles are much stronger in END-COMPRESSION than transversely.
tion urged against this construction is that it is wasteful of mortar
t to get the edges of the blocks properly bedded. They do require
re mortar, but the second objection is not serious, for, if the blocks
a proper bevel, the tighter they are set the stronger the arch."*
lual blocks in the END-CONSTRUCTION are commonly made rectan-
ape, advancing by 1 in from 6 to 15 in in depth. The length and
of the blocks may be varied, but the standard size is 12 in for both
The number of partitions or webs in the blocks varies with the
blocks and also with the strength desired. The 6, 7 and 8-in blocks
e two vertical partitions and one horizontal partition, or one vertical
rizontal, for blocks 8 in wide. The 10 and 12-in arches may have
or two horizontal partitions. Arch-blocks over 12 in deep should
e at least two horizontal partitions. In the strongest blocks the
ibout 3 in square. "The arch-blocks must be set end to end in
irses from beam to beam, and cannot be set breaking joints, as in
NSTRUCTION method."* So that if one block fails, the rest of the
e width of that block, is dependent for its strength on the adhesion
idual blocks to those adjoining.

ss of Web. This should be at least ¾ in for porous and ½ in for
tiling. The thicker the webs the greater will be the strength and

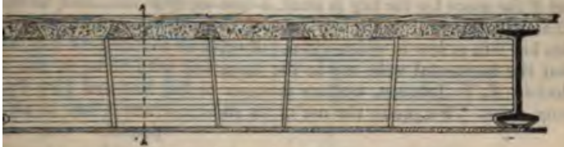


Fig. 21. Flat Tile Floor-arch. End-construction

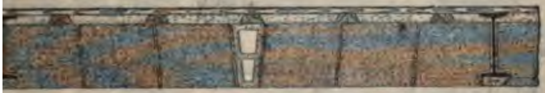


Fig. 22. Flat Tile Floor-arch. End-construction

e of the arch. The end-joints are always beveled, as in Figs. 21
ends being parallel; thus all the intermediate blocks are made with
e.

Skew-backs. An end-construction arch may have SKEW-BACKS
e same blocks, with notches in the ends of the blocks to fit over the
ges of the beams, as in Fig. 21. It is generally considered that the
ction skew-back is much stronger than the side-construction skew-
account of the large amount of mortar lost in the voids and the
f obtaining an even bearing with end-construction skew-backs,
ecause of the greater facility with which the side-construction skew-
e used, contractors generally prefer to use the latter; and this has
the COMBINATION-ARCH, shown in Fig. 23. But a more important
ising side-construction *skew-backs with end-construction arches,* **is**
protection against fire that they afford to the beam or girder. To

---

* *Bevier.*

develop the necessary strength, side-construction skew-backs should large sectional area and a sufficient number of partitions, following mately, the lines of thrust. With any form of skew-back the recess beam-flange should be of ample width, so that when the tiles are set the tecting flanges on the skew-backs will not touch the bottom of the beam

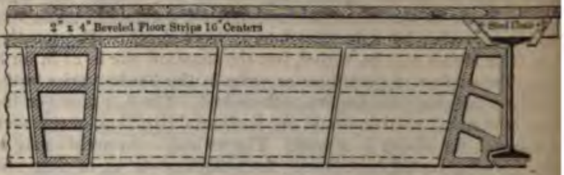


Fig. 23. Flat Tile Floor-arch. Combination End and Side-construction

will be at least ¼ in below them. Many varieties of side-construction backs are made to meet all possible conditions.

Keys. Both end-construction and side-construction KEYS are used end-construction arches, the choice of the key depending principally length. If the span of the arch is such that the standard intermediate require a key 6 in or more in width, the END-METHOD KEY is used, as in but if the space for the key is small, a SIDE-METHOD KEY, such as shown 22 and 23, is used. As the key is almost entirely in compression, a side-tion key 6 in or less in width will usually give all the strength required, that the horizontal webs are in the same line with those in the inter blocks. E. V. Johnson, western manager of the National Fire Proofing pany, says: "We prefer the use of an end-construction key in all cases possible. Our custom use side-construction spaces of 6 in and end-construction keys spaces. When using keys we insert a ¾-in slab between the end tile."

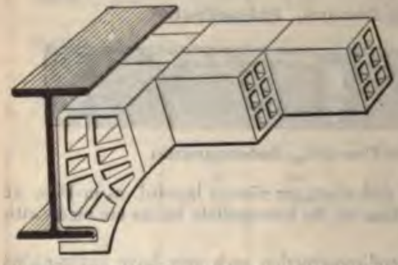


Fig. 24. Raised Side-construction Skew for End-construction Tile Floor-arch

Raised Skew- Where flat arches between 18, 20 or it is necessary either RAISED SKEW-BACK have a large space top of the tile arch must be filled in Raised skew-backs are preferable to a hollow space above the tiles and than concrete filling. They are often used for roof-arches, because for pose it is seldom necessary to make the arches as deep as the beams, top must be about on a level with the beams. Raised skew-backs always made on the side-construction principle. Figs. 24 and 25 show forms of raised skew-backs for end-construction arches.

Flat Versus Paneled Ceilings. In connection with the raising arches above the bottom of the beams or girders, J. K. Freitag calls to the advantages of FLAT CEILINGS, as follows: "Flat, unbroken o be preferred to any type of terra-cotta arch which may require a effect due to the projection of the girders or beams below the main ne." A perfectly flat ceiling reflects more light, makes a better-lighted d deflects the heat. Paneling forms pockets for the retention of heat e and greatly increases the exposed area.

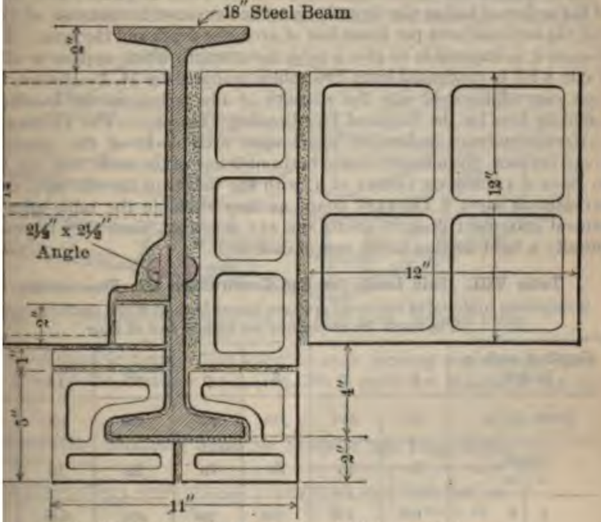


Fig. 25. Raised Tile Floor-arch and Girder-protection

-Arches and Beams of the Same Depth. A deep block makes a ronger floor than a shallower one, and for the same depth of beams and cheaper floor. A 12-in arch weighs less per square foot than a h with 2 in of concrete filling; and it costs less.

, Span and Weight. The MAXIMUM SPANS for different depths and AGE WEIGHTS per square foot of this type of arch, set in place, are as

'II. Maximum Spans for Flat Tile Floor-Arches of Different Depths and Weights

Depth of arch, in	Maximum span, ft in	Weight per sq ft, lb
6	4 6	29
8	5 6	31
9	6 0	32
10	6 6	33
12	8 0	39
15	9 0	46
16	10 0	50

ights per square foot, as given by different manufacturers, vary greatly, bt, to the character of the material used and to the thickness of the webs.

The DEPTH OF ARCH most frequently used is 10 in, the girders bein
to use 10-in I beams for joists spaced from 5 to 6 ft apart. As a rule the
of the arch should be about equal to the depth of the beam, as it is just al
cheap and much better construction to use deeper tiles and less concrete

Safe Loads for End-Construction Tile Floor-Arches. The str
of flat arches of hollow tile depends upon the CRUSHING RESISTANCE of the
rial, the sectional area per linear foot of arch, the depth and the span. Fo
reasons it is impossible to give a table for strength which applies to all
Table VIII is condensed from two tables prepared by H. L. Hinton, w
gone very elaborately into the strength of tile arches, in the handbo
pared by him for the National Fire Proofing Company. The values gi
END-CONSTRUCTION arches are based upon arch-blocks of the cross-s
areas, per foot, given in the second horizontal line of the table, and are
to have a FACTOR OF SAFETY of 7, with the weight of the tile only, d
Mr. Hinton says: "The SAFE LOADS as they stand in the table affor
general statement of SAFE LOADS FOR ALL SECTIONS, since they repres
cifically a light section in the case of each arch."

Table VIII. Safe Loads for End-Construction Tile Floor-arches.

Semiporous material of sectional area per linear foot, as given in the seco
The loads are in pounds per square foot of floor

Depth of arch in inches	6	7	8	9	10	12
Areas, sq in	310	340	370	400	430	490
Spans, ft in	lb	lb	lb	lb	lb	lb
4 6	196	254	319	391	470	648
5 0	155	202	254	312	376	519
5 6	......	163	206	254	306	424
6 0	......	......	170	209	253	352
6 6	......	......	141	175	212	295
7 0	......	......	......	147	179	251
7 6	......	......	......	......	153	215
8 0	......	......	......	......	......	185

* This table is condensed from two tables prepared by H. L. Hint

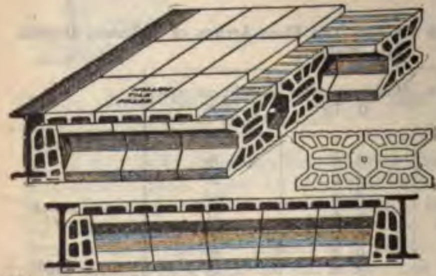
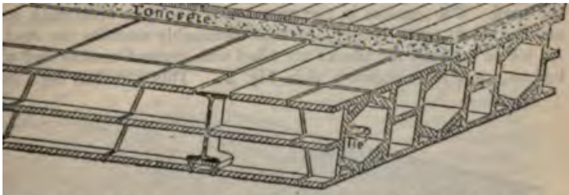


Fig. 26. Excelsior End-construction Tile Floor-arch. Side-skew

**Patented E
struction Tile
Arches.** Figs. 2
show two variati
type of arch inve
patented by E. V
when manager
Pioneer Co
Chicago, Ill. Th
manufacture and
arch, in certain
was granted to th
Company; also
Maurer & Son, N
City, and to a co
Haydenville, Ohio. The original shape of the arch-tile is illustrate



. 27. Johnson End-construction Tile Floor-arch. Original Form

of suitable depth. The LIMIT OF SPAN, WEIGHT PER SQUARE FOOT DAD of the Excelsior arch (Fig. 26) is given by Maurer & Son as

ble IX. Maximum Spans for Excelsior Tile Floor-Arches

pth of arch, in	Limit of span, ft	Weight per sq ft, lb	Safe load per sq ft, lb
8	5 to 6	27	300
9	6 to 7	29	350
10	7 to 8	33	300
12	8 to 9	38	350

er Company has made arch-blocks as deep as 20 in and as heavy as ft. This company and Henry Maurer & Son use semiporous ma- e arch-blocks. It should be noticed that the arch made by the an END-CONSTRUCTION SKEW-BACK, while the latter uses a SIDE- ON SKEW-BACK. The Pioneer Company formerly used the side- skew-back, but found that when arches of this type were tested on the skew-backs were almost invariably the parts which failed; doption of the end-construction skew-back. Henry Maurer & Son, of construction. On the other hand, however, they are undoubtedly pensive than cinder-concrete floor-construction, because of the mat and the increased height of the building due to thicker floors.

The Herculean Arch.* These floor-arches are built of semipor cotta blocks, 12 by 12 in on top and varying from 6 to 12 in in depth, to the span and load. In the sides of the blocks are grooves to rece 1½ by 3/16-in T bars. The blocks are laid end to end the entire len span, with a bearing of from 4 to 6 in on the walls or girders, prese continuous grooves, which are filled with cement mortar, and into T bars are then inserted. The T bars must, of course, extend the full the span. The grooves in the next course are then filled with cemen and the blocks pushed into place, thus thoroughly covering the steel wit All joints between the blocks are filled with cement mortar and the b laid to break joint endwise, as in Fig. 28. This floor has been used i

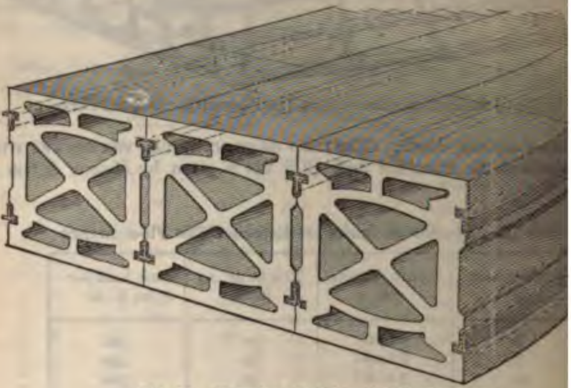


Fig. 28. Herculean Reinforced, Tile Floor-arch

varying from 19 to 23 ft. The WEIGHT per square foot given for cotta blocks and steel T bars is 26 lb for blocks 6 in deep, 33 lb for 8 42 lb for 10-in blocks and 51 lb for 12-in blocks. The manufacturer the SAFE LOADS for this construction as follows:

For a 12-in arch with a 20-ft span, 400 lb per sq ft.
For a 10-in arch with a 16-ft span, 400 lb per sq ft.
For a 8-in arch with a 12-ft span, 150 lb per sq ft.

The CHIEF ADVANTAGE of this construction is said to be its low cos pared with the cost of systems equally fire-proof and requiring st every 6 or 8 ft. It is particularly well adapted to buildings with mas and partitions, as in such buildings little or no structural steel is requir floor-construction affords, also, an unusually smooth undersurface reducing the cost of plastering. No TIE-RODS are required for this floo

The Johnson Long-Span Flat Floor-Construction. This REI TILE FLOOR was invented by E. V. Johnson, and is now controlled an by the National Fire Proofing Company. Its general construction is flat centering is first erected, and over this is spread a layer of d-cement mortar about ¾ in thick. On top of this mortar is laid BRIC containing steel rods varying from ¼ to ½ in in diameter, the span, and spaced from 2 to 8 in, center to center. Another same mortar is then spread on top and hollow tiles, from 3 to 12 in in ding to the span, are then set in the mortar and laid so as to BREAK o form continuous rows from one support to the other. A layer also, about 2 in thick, is usually spread on top of the tiles. Fig. 29

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* Patented and manufactured by Henry Maurer & Son, 1898 and 190

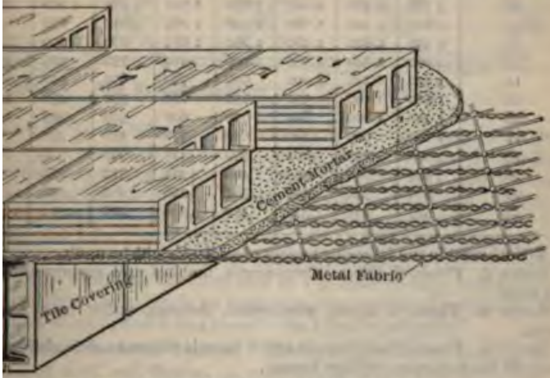


Fig. 29. Johnson Reinforced, Tile Floor-arch

neral method of construction of this system, but without the rods, serted in place as the fabric is used. For short spans the fabric without the rods. This system differs from the flat concrete system ubstitution of hollow tiles for the concrete in the upper portion of e strength of the floor depending upon the REINFORCEMENT and of the cement mortar to the steel and tiles. As the tiles are h on the bottom and top with concrete, the FIREPROOFING PROP- s measured by the resistance of the concrete and not by that of the have shown that the ADHESION of the mortar is perfect and that it high temperature without injury. This construction can be used up to 25 ft, the most ADVANTAGEOUS SPAN being about 16 ft. The square foot, including the fabric and the cement on the bottom and , but not on top of the tile, is as follows:

f tile, inches............	12	10	9	8	7	6	5	4
per square foot, in pounds	60	55	45	42	37	34	26	24

ete above the tile should be figured at 12 lb per sq ft for each inch The STRENGTH of the floor, with 1 in of 1 : 3 Portland-cement of the tiles, is given as follows:

Table X. Ultimate Strength of the Johnson Floor-Const

Span in feet	Thickness of tiles in inches						
	12	10	9	8	7	6	5
	Ultimate strength in pounds per square fo						
10	3 375	2 580	2 140	1 850	1 525	1 265	1 000
11	2 800	2 340	1 780	1 536	1 264	1 052	832
12	2 350	1 800	1 480	1 280	1 064	880	700
13	2 000	1 540	1 265	1 100	910	752	595
14	1 730	1 325	1 100	950	780	650	510
15	1 500	1 160	950	830	680	590	450
16	1 320	1 010	840	720	600	500	395
17	1 180	900	740	640	578	440	350
18	1 020	795	664	570	473	392	310
20	844	645	535	462	381	314	250
22	700	536	445	384	316	263	208
24	587	450	370	320	266	220	....

With this table the following factors of safety should be used:

Factor 4. Floors of offices, school-rooms, hospital and asylum-w
and roofs.

Factor 5. Floors of stores, warehouses, theaters, public halls
rooms.

Factor 6. Floors of buildings in which there is vibration of machin
there are loads causing sudden impact.

A section of this floor, 16 ft square, supported on walls around
and loaded over its entire area with a total uniformly distributed
per sq ft, showed a DEFLECTION of ¼ in, full; and with a load o
ft, a DEFLECTION of ⅛ in, scant. The ADVANTAGES of this syster
as noted for all LONG-SPAN FLAT SYSTEMS. It can be used to spe
for roofs and for buildings so divided by masonry partitions t
do not exceed 25 ft. For such buildings very little, if any, stru
required.

The New York Reinforced-Tile Floor-Arch. This arch

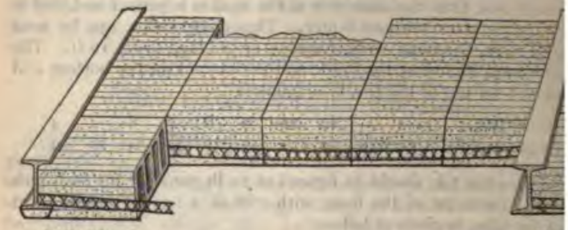
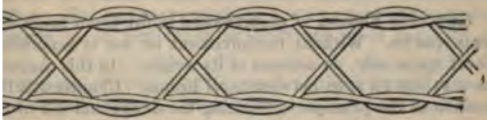


Fig. 30. New York Reinforced, Tile Floor-arch

designed by P. H. Bevier, of the New York City branch of the
Proofing Company, for use "when a light and cheap but strong f
tion with a flat ceiling is required, and is particularly adapted

/hen light floor construction with deep beams is necessary
etting the blocks level with the tops of the beams and using
ng, or by omitting the ceiling a panel effect is obtained.
are used the blocks are set level and 1 in below the bottom
cinder concrete or dry cinders is used to level up to the
The WIRE TRUSS REINFORCEMENT, Fig. 31, used in this



. Wire Reinforcement for New York Floor-arch

the building in reels, and is cut to proper lengths on the
embedded in Portland cement mortar between the blocks,
both against rust and fire. The open-work construction
ables the mortar to flow freely all about it and the joint
led between the blocks and the wire perfectly embedded.
span, and 8-in arch for 7-ft 6-in span, have been tested
ldings, of New York, and accepted for live load of 150 lb
York arch has been used in a number of large buildings in
sts were made to determine the ULTIMATE STRENGTH of
span, and it was found to be 1 600 lb per sq ft."

Tile-Arch System. This is a method devised by R.
York and Boston, of constructing floors, partitions, stair-
of THIN TILES, 1 in thick, about 6 in wide and from 12 to
l together in Portland-cement mortar so as to make one
rs are built by spanning the spaces between the girders
aults, or domes, constructed of two, three, or more thick-
he number of thicknesses depending upon the dimensions
s. In its best application, steel is used in tension only in
place of steel girders, tile girders are constructed of the
rever steel is used it is embedded in the masonry construc-

otable buildings in which this system was used is the Boston
ling, completed in 1895. Some of the later important
Cathedral of St. John the Divine, New York City; the
tol Building, St. Paul, Minn.; the Girard Trust Company's
a; the Chicago and Northwestern Railway terminal
Pennsylvania and the New York Central Railroad ter-
ork City; and the Hall of Fame, University of New York,

he wide spans that can be safely used with this system of
the Cathedral of St. John the Divine in New York City.
crypt, measuring 56 by 60 ft, with no interior supports,
a safe load of 400 lb per sq ft, was constructed on this
a VAULTED CEILING is desired this form of construction
pted for use. Floors built in this way have been tested
of the New York City Building Department up to 3 700 lb
10 ft. When used between **I** beams the only steel beams
nning from column to column. Architects contemplating
of construction are *advised to consult the R. Guastavino*

short spans only, on account of its weight. In this chapter it
as a FLOOR-FILLING between steel beams. Chapter XXIV is
cussion of the principles governing the design and use of reinfo

Advantages of Reinforced Concrete for Floor-Constru
many ADVANTAGES are claimed for reinforced concrete over
the principal advantage is that of economy, taking into ac
both the steel framework and the filling between. The oth
vantages are less weight per square foot of floor (usually bu
case), adaptability to irregular framing and rapidity of cons
in the immediate locality of the tile-factories, fire-proof floor
usually be placed at less expense than is incurred in setting flo
and when the spans permit the use of cinder concrete, the c
lighter than those of the tile, when both floors have the same
of the long-span tile-systems, on the other hand, are much l
of the concrete floors that are now being built. The material
construction of reinforced-concrete floors are readily obtain
locality, no specially prepared material is required, except
special forms of reinforcement, and the work can be done a
unskilled labor. Less capital is required for concrete work
constructions, and no material need be carried in stock duri
except tools, mixing-machines, old centering, etc. That the
are real is sufficiently proved by the immense amount of re
now under construction throughout the world. Wherever a
finished, cement surface, reinforced-concrete constructions
cheaper than any tile-system, because in the former, the enti
to give strength, while with the flat-tile arches it merely i
weight.

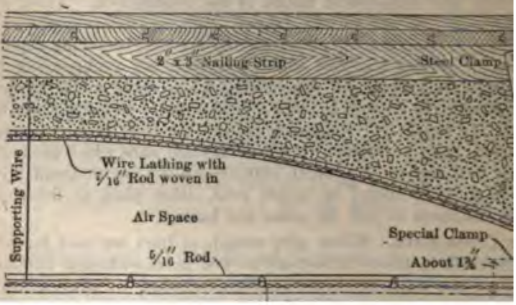
Disadvantages of Reinforced Concrete for Floor-Co
decided DISADVANTAGE connected with concrete floor-constru
ference in a large measure with the progress of other parts of

inforcement. While steel in small sections is used almost reinforcement, there is a great variety in the shape and char- l employed. Different forms of reinforcement are described Chapter XXIV. All of them may be used, and most of them ed in floor-construction. In addition to those forms discussed t are not readily adapted to beam-construction are used in . Such are the metal fabrics described farther on under es of construction. The proper position for the reinforcement uction is that in which it will take the tensional stresses, labs, near the lower surface. The most logical form is that of greater number of small rods or bars is preferable to a smaller ones, because the proportion of the area of adhesion between to the sectional area of steel is greater in the former case. parently attained in systems in which wire fabrics are used. tage in the use of the smaller reinforcement is the greater pos- sion and consequent failure of the construction. There is a tage in the use of wire fabrics; they are easily displaced in cing of concrete, either getting too low and becoming exposed on, or getting too high with a corresponding weakening of the detail that must be remembered when using metal fabric is ust be large enough to allow a good bond to be formed between e and below it. Reinforcements in the form of bars set ver- crete have a tendency to shear through slabs which are under he best and most logical reinforcement for fire-proof floors ½ to ¾-in round or square rods, either plain or deformed, g distances to suit the spans and loads.

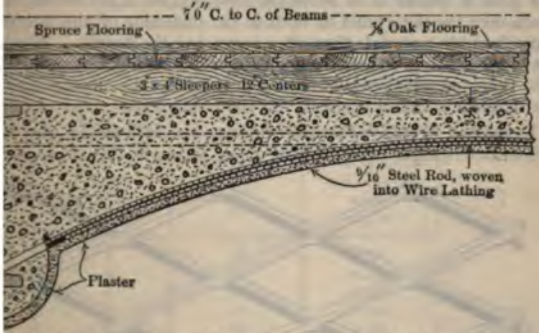
Cross-Bars. Where wire strands or bars are used for rein- ssential to have cross-bars as well as transverse tension- hen the loads are heavy and concentrated, or when a heavy slab the concrete will crack between the carrying bars. This spaced 40 in apart. The structure was one year old when tes

While there are several patented arched-floor systems, a pl can be built by any one; and if reinforcing is desired for wide or bars, small tees or channels, and various forms of netting m out infringing on any patents. The principal advantages of th systems lie in the matter of economy in putting the arches concrete arch, considered as a monolithic construction, if concrete, is superior to the brick arch. The cinder-concrete only in point of strength. Such an arch should be at least crown, and the rise should be not less than one-eighth the spa of such an arch for ordinary cinder concrete is about the same segmental-tile arch of the same span, as given in Table V. All whether of concrete or tile, require tie-rods between the beam thrust of the arches. (See page 871.)

The Roebling Arch Floor-System. This system is now s that it requires but a brief description. It has been used in m



rogress continuously. The centering is made of the proper size the factory, so that it is readily placed in position. This cen- antages, aside from the saving over wooden centers, and the which it can be put in place. It allows the superfluous water to e concrete as soon as it is in position and it also forms a valu- a from falling for the workmen, as it is sufficiently strong in


7′0″ C. to C. of Beams
Spruce Flooring
⅞ Oak Flooring
3 × 4 Sleepers 12 Centers
5/16″ Steel Rod, woven into Wire Lathing
Plaster

g. 33. Roebling Segmental Concrete Floor-arch. Type 2

n a considerable load. Sections of the Roebling-arch floor have ithout failure with loads of from 1 400 to 4 100 lb per sq ft. quire a corresponding depth at the haunches, as the clear rise of ypes 1, 2 and 3 should be 1½ in per ft of span. In an 18-ft rise above the beam-flange was 16 in. For a 14-ft span between e rise would be 14 in.

g table, prepared by the Roebling Construction Company, gives square foot for different spans:

I. Weight per Square Foot of the Roebling Floor-Arches

crete side ns,	Maximum spacing of steel floor-beams (independently of size of beams), ft in	Thickness of crown at center of arch, in	Weight per sq ft including only concrete and wire, lb
	4 0	3	33
	4 6	3	34
	5 0	3	36
	6 0	3	41
	7 6	3	47

given in the table are for concrete to the level indicated in the with a 3-in crown and for all wire construction, including arch- and lathing for ceilings.

ced Floors. *These floors consist of slabs of concrete, varying rding to the span and load, constructed between the steel* incombustible material, thus often increasing the weight.

set at the top flanges, there is a paneled ceiling, unless a hung c
The types of reinforcement used are generally the distingui
the different so-called SYSTEMS, but the principles involved ar

Expanded Metal. This material is now so well known tha
a brief description. The diamond mesh shown in Fig. 34 is

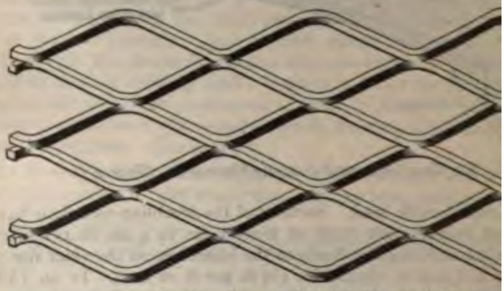


Fig. 34. Expanded Metal, Diamond Mesh

struction. For this purpose the 3-in mesh is used, the size c
designated by the width of the diamond-shaped spaces. I
8 or 12 ft long, and from 3 to 6 ft wide, according to the w
It is made from a soft, tough steel of fine texture, varying i
No. 13 to No. 6, Stubbs gauge. When used between **I** bea
reinforcement, the spans usually vary from 6 to 8 ft, alth
wide between beams have been constructed.

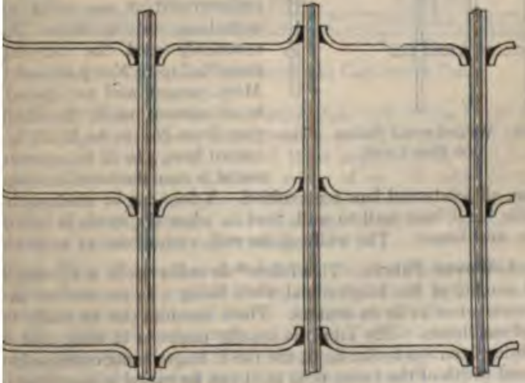
Concrete and Expanded-Metal Floor-Construction.
styles of floor-construction possible with expanded-metal r

er sections at greater intervals; an increased ultimate strength tic limit, due to the method of manufacture, thus combining es of a low-carbon steel with a high ultimate strength; and a nd with the surrounding concrete. The standard sizes offered by ted Expanded Metal Companies and the Northwestern Expanded ny are in accordance with a decimal variation in cross-section, 30, 0.35, 0.40, etc., sq in per ft of width. The designations of cate the cross-sectional areas per foot of width, thus: 3-9-20 mesh, No. 9 gauge plate and a cross-sectional area of 0.20 sq width. The Youngstown Iron & Steel Company and the Gen- ing Company offer from eight to ten sizes of expanded metal with ent to take care of the needs of concrete-floor designs.

Table XII. Properties of Rib-Metal

Size-number	Width of sheet in	Area of metal per foot of width sq in
2	16	0.54
3	24	0.36
4	32	0.27
5	40	0.216
6	48	0.18
7	56	0.154
8	64	0.135

. The Trussed Concrete Steel Company of Detroit, Mich., is g a steel reinforcement for concrete floors consisting of a series



Area of rib, 0.09 sq in
Ribs spaced 2, 3, 4, 5, 6, 7 and 8 in

Fig. 36. Rib-metal Reinforcement for *Concrete Floors*

or main tension-members rigidly connected by light cross-ties the same sheet of metal in the form of a mesh. (*Fig. 36.*) It

is manufactured from medium open-hearth steel in seven sizes of me
5, 6, 7 and 8 in, and in lengths up to 18 ft. It is supplied in either fla
sheets and longer lengths and special sizes of mesh can be provided.
of sheet is governed by the size of mesh, there being nine bars or ri
sheet.

Welded-Metal Fabric. The Clinton Wire Cloth Company man
welded fabric or mesh which has been extensively used in the United
a reinforcement for concrete co
of all kinds. Fig. 37 shows th
style of the fabric, the meshes are
which can be varied indefinitely,
from a 1-in mesh. The advantag
for this fabric as a reinforcement
construction is that the carrying
be varied, both in size and spacin
the necessary area for any given
span, and the distributing or cr
also, may be varied in the same
direction of the wires coincides
LINE OF STRESS, so that there is
ency to distort the rectangle of
The cross-wires, being welded to
ing wires, are rigidly held in
prevent the latter from slippin
concrete. The claim is made
elongation that takes place in th
wire under the stress of heavy
divided along the carrying wire
the cross-wires occur, instead
concentrated at one point as
with loose rods or wires. In
commonly used the carrying
from No. 10 to No. 3 in size (W
Moen gauge), and are spaced f
in on centers; while the distrib
vary from No. 11 to No. 6 in si
spaced from 3 to 12 in on centers
metal is manufactured in long r
its use, all joints and laps are avoided. A floor can be made with a
metallic bond from wall to wall, that is, when the mesh is laid ov
of the steel beams. The width of the rolls varies from 48 to 86 in.

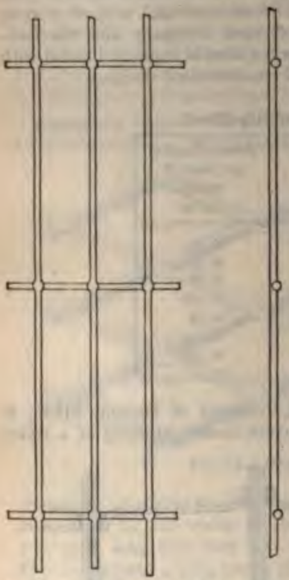


Fig. 37. Welded-metal Fabric. Clinton Wire Cloth

Lock-Woven Fabric. This fabric* is made up in a rectangula
usual spacing of the longitudinal wires being 3 in on centers and
transverse wires 12 in on centers. These spacings can be easily var
special conditions. The fabric is usually made 54 in wide and co
containing from 150 to 600 lin ft, the 150-ft length being commonly u
the usual width of the fabric is 54 in, it can be varied in multiples of
18 in up to a maximum of 54 in. The longitudinals or carrying
fabric are held in place by wrapping around them the transverse wir
in Fig. 38. The longitudinal wires can be furnished in sizes varying
to No. 7; the transverse wires are usually No. 14 or No. 12. The

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[*] Controlled by W. N. Wight & Company, New York Cit

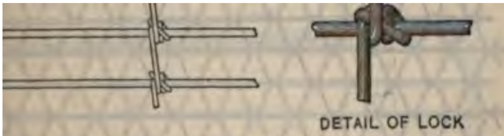
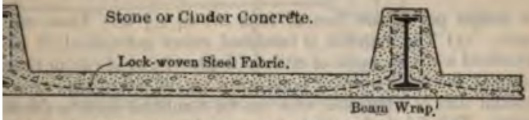


Fig. 38. Lock-woven Fabric

o end of a structure, and the wires are so placed that they lie INES OF STRESSES which they are called upon to carry. The f construction for floor-slabs and roof-slabs is similar to that for expanded metal. Where a flat ceiling is desired the type hown in Fig. 39 is very useful. Both of these types have y the Bureau of Buildings of the City of New York on spans



. Concrete Ceiling-slab Reinforced with Lock-woven Fabric

ling 6 ft, for live loads running from 130 to 330 lb per sq ft; 7 ft, approvals have been given up to 175 lb per sq ft, and on to 150 lb per sq ft. All of these approvals are based upon r concrete and on a factor of safety of 10. In addition to its truction of floor-slabs and roof-slabs, the fabric is suitable for lls, sewers, penstocks and tanks, and in all other places where ment can be used to advantage.

indicated in Figs. 29, 35 and 39. The longitudinal wires in Tri
invariably spaced 4 in on centers, but the diagonal wires may b
2 or 4 in apart. The manufacturers can furnish eighty-eight
giving variations in the cross-sectional area of the longitudi
about 0.038 sq in to about 0.380 sq in per ft in width of the fabri

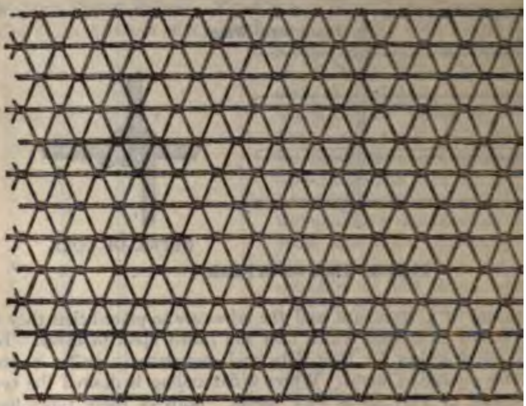


Fig. 40. Wire-fabric Reinforcement, Triangular Mesh

in weight per square foot of from 0.2 to 1.5 lb. These styl
ways. (1) The material is furnished either galvanized or pla
gitudinal wires are made of either a single wire or of two or thre
and (3) the cross-wires or bond-wires are of either No. 14 or
Special sizes of additional area can be furnished upon applicat
pany. This fabric is said to have an ultimate strength of not
lb per sq in.

Patented Systems of Flat Floor-Construction. The follo
floor-construction, described in the following paragraphs, while
same general principles as those already described, are paten
used only by the patentees.

Roebling Flat Floor-Construction. This system was int wever, is not desirable when the **I** beams are more than 7 in deep. istance between the steel beams is greater than 9 or 10 ft, the are bent downward so as to give a sag of about 2 in or more at the span, as in Fig. 42, the spacers being used as in Fig. 41. This

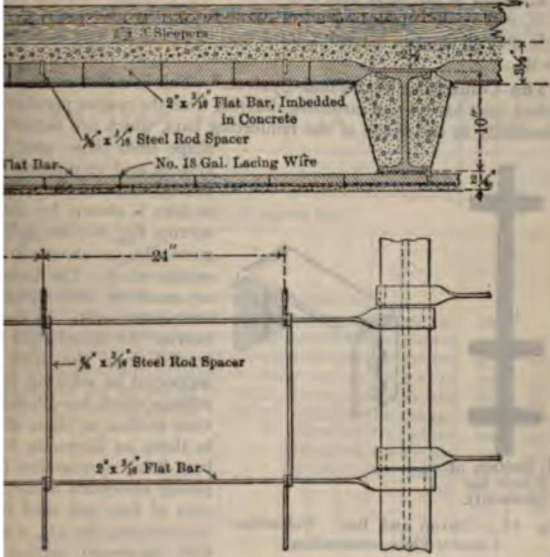
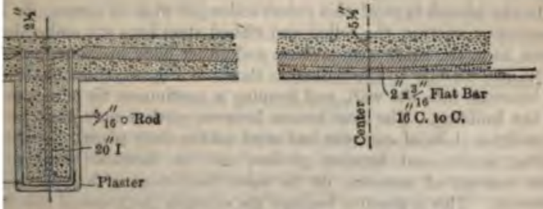

2 x 3 Sleepers
2½"
3½"
2" x 3/16" Flat Bar, Imbedded in Concrete
⅜" x 3/16" Steel Rod Spacer
lat Bar
No. 18 Gal. Lacing Wire
10"
2½"
24"
⅜" x 3/16" Steel Rod Spacer
2" x 3/16" Flat Bar

Fig. 41. Roebling Flat Concrete Floor-construction

Type 5 and has been successfully used in spans up to 22 ft. Under litions, however, considering both the steelwork and the fireproofing, omical results are obtained when the girders are spaced from 14 to With this system a suspended ceiling is not necessary or desirable.



ig. 42. Roebling Flat Concrete Floor-construction. Type 5

used with this system is composed of high-grade Portland cement sand, and clean cinders, mixed ordinarily in the proportions is floor-system is particularly adapted to public buildings, offices, hospitals, hotels, residences, etc., or where there is no great

weight to be supported, and the fire-hazard is not as great as in stores,
etc. It can be successfully adapted, however, to stores and wareho
requires shorter spans and heavier construction. The Roebling Con
Company claims that Type 1 has a safe carrying capacity, with a factor
of 4, of 200 lb per sq ft with a span of 8 ft, and that Type 5, with
16 ft, will safely support a load of 100 lb per sq ft. A section of floor
wide and 16 ft span, carried a total load of 17 250 lb with a deflectio
7/16 in.

The Columbian Flat Floor-System.* This is a flat concrete sys
ribbed steel-bar tension-members, differing from the system previously
principally in the shape of the reinforcing bars, which are entirely di
shape from those used in
system, and very much dee
general shape of the 1, 2, 2½
in bars is shown by the ho
stirrup, Fig. 43; the larger s
more ribs, as shown in the
section at *A*. The Columbi
are made in two styles, LO
and SHORT SPAN. In the SH
SYSTEM the ribbed-steel bars
pended from the steel bea
supported on edge by mean
stirrups which have holes of
cross-sections as those of the
in them, as shown in Fig.
bars being surrounded by
pletely embedded in concrete
sizes of bars are used for
construction, the 2½, 2 and
their maximum spacing bei

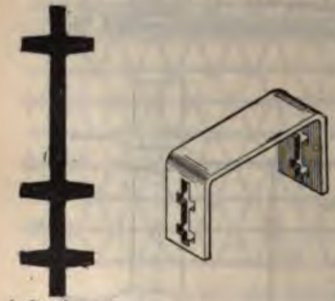

A, Section of 5-inch bar (reduced).

Fig. 43. Stirrup and Bar. Columbian Concrete Floor-construction

The carrying capacity of this floor is given in Table XIII. The most
ical spacing of floor-beams for this type is usually 6 ft for hotels, ap
houses and office-buildings, using 1-in bars, and from 6 to 9 ft fo
floor-loads, using 2 and 2½-in bars, depending upon the load requir
carried. Economy is claimed for this type of construction in t
channels are not required, and beams may be spaced up to 9 ft,
center.

In the second type of this construction, or what is commonly know
LONG-SPAN SYSTEM, the rolled and ribbed steel bars are embedded in
crete, as in the short-span system, and either hung in specially formed
or framed directly to the beams, as shown in Fig. 44, these bars being
at intervals into the wall, and forming a continuous tie across the en
of the building. The floor-beams between girders may thus be omi
monolithic slabs of concrete and steel taking their place. In this wa
ceiling is obtained between girders, and an increased head-room
same amount of masonry, or the same head-room with a decreased
masonry. This is possible because the extreme thickness of this floor-
tion on a span of 20 ft between beams, is but 6½ in, whereas for a span
with a lighter load, the thickness of concrete may be reduced to 5
sizes of bars used in this type of construction are 3½, 4½, 5 and 6 in,
respectively 5, 5¾, 6½ and 7½ in of concrete.

---

* Controlled by the Columbian Concrete-Steel Bar Company, Pittsbu

II. Safe Loads in Pounds per Square Foot for Columbian Floor-Construction

STONE CONCRETE

ar in wt ar ft	2½ × ⅛ in 1.65 lb									3½ × 13/64 in 3.8 lb						4¼ × ¼ in 5.3 lb	
g of nches	24			20			15½			24			21			24	
f steel ft of lbs	0.83			0.98			1.3			1.9			2.2			2.65	
ess of nches	4	4½	5	4	4½	5	4	4½	5	5	5½	6	5	5½	6	5¾	6½
feet	Pounds per square foot																
	560	...	...	672	...	...	...	...	...	...	...	...	...	...	...	......	......
	360	440	...	432	528	...	480	...	...	...	...	...	...	...	...	......	......
	250	305	335	300	366	402	335	390	...	...	...	...	...	...	...	......	......
	185	225	245	222	270	294	245	285	325	550	610	660	...	...	...	......	......
	140	170	185	168	204	222	185	220	250	420	465	510	520	580	640	680	......
	110	135	150	120	162	180	150	170	195	335	370	400	420	460	500	540	780
	90	110	120	110	120	144	120	140	170	270	295	325	340	370	400	435	620
	75	90	100	90	110	120	100	125	150	220	245	270	280	310	335	360	415
	60	75	95	75	90	114	85	100	130	190	205	225	245	260	280	300	350
	...	...	70	...	75	95	...	80	100	160	175	190	200	220	240	260	300
	...	...	...	...	...	70	...	...	80	140	150	165	175	190	210	225	255
	...	...	...	...	...	...	...	...	...	120	135	155	150	165	180	190	225
	...	...	...	...	...	...	...	...	...	100	120	135	110	145	160	170	195
	...	...	...	...	...	...	...	...	...	...	...	...	...	...	125	155	175
	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	100	125
	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	60	85

CINDER CONCRETE

r in wt r ft	1 × 3/16 in 1.1 lb					2 × ⅛ in 1.45 lb					2½ × ⅛ in 1.65 lb								
of ins	24		20			24		15			24			20			15½		
eel of lbs	0.55		0.65			0.73		1.15			0.83			0.98			1.3		
ss ins	3	3½	3	3½	4	3½	4	3½	4	4½	4	4½	5	4	4½	5	4	4½	5
eet	Pounds per square foot																		
	225	265	300	375	450	400	420	425	475	525	450	500	575	640	600	690	650	700	...
	160	200	215	265	325	300	330	325	350	400	350	370	440	470	444	528	550	575	650
	90	130	150	190	215	210	240	230	255	300	250	270	300	350	324	360	430	460	480
	60	70	80	115	140	140	165	160	175	200	170	185	210	204	222	252	250	275	325
	...	...	...	...	85	85	100	90	120	135	110	130	140	132	156	168	160	200	225
	...	...	...	...	...	...	60	...	80	100	70	100	110	84	140	140	105	150	175
	...	...	...	...	...	...	...	...	...	75	...	70	80	...	84	96	75	110	135
	...	...	...	...	...	...	...	...	...	...	...	...	*60*	...	...	*72*	...	*90*	*110*
	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	80

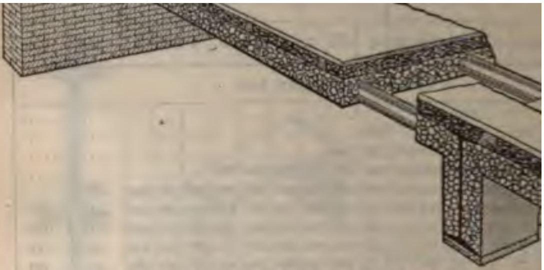
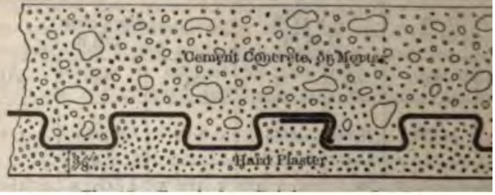


Fig. 44. Columbian Long-span Concrete Floor-constru

hours' duration made on this system in Boston,* this floor o
3 in, carried 1 650 lb with a deflection of only $1\frac{3}{8}$ in. Before th
to show any sign of failure, the loading had to be stopped b
of the test-hut which carried the floor began to crack.

Dovetailed Corrugated Sheets. Ferroinclave. Sheets c
gated so as to form dovetailed grooves have been used as a reinfo



-side should be more or less dislodged by the streams of water, placed, at a very slight expense. Another advantage in the use ve for roofs is that a building can be covered and made water- nost severe winter weather and the cement applied during the fol- .

re is made in sheets 20 in wide and up to 10 ft long, and it is usu- 4 gauge. For roofs it is attached to purlins in the same way that s attached, the most economical spacing of the purlins being 4 ft to center, which accommodates sheets 10 ft long and leaves an end- For the cement top coat on roofs, a mixture of one part Portland ro parts sand, applied to a thickness of ⅜-in above the top of the icient. For floors a rich, gravel or crushed-stone concrete should be kness being governed by the span and the loads to be supported.

ing table shows the ultimate strength of No. 24 Ferroinclave with knesses of concrete, as determined by actual tests with sheets 20 in -ft 10½-in span:

inches of 1 : 2 mortar above	1½	2	2½	3	3½	4
ngth in lb per sq ft for a span	615	915	1 220	1 560	1 860	2 120

safety of 6 should be ample for ordinary loads.

illion square feet of Ferroinclave have thus far been used for floors, side walls. It is especially adapted for the walls, roofs and floors ufacturing plants, and may be used to advantage for partitions, -treads, vats, water-closet partitions and fire-proof doors.

Multiplex Steel Plate. Fig. 46 shows a section of a corrugated nanufactured by the Berger Manufacturing Company, Canton, r and roof-construction, the plate being an invention of G. Fugman.

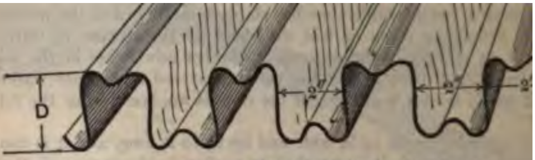


Fig. 46. Berger's Multiplex Steel Plate

While this floor has several practical advantages, it cannot thoroughly fire-proof, because the metal is exposed on the bot a plastered ceiling on the under side, the iron would probably by any ordinary fire before the latter could be controlled.

Permanent Centering. Numerous forms of sheet-metal fa those just described, have been developed in recent years for t forcements. They consist, generally, of steel plates pressed int ribs, variously spaced, between which the metal is stamped o

ed by expanded metal. The sheets are 28 in in width and come in
o 14 ft. Self-Sentering is made of No. 24, 26 and 28-gauge metal.
age 891.)

Hy-rib, controlled by the Trussed Concrete Steel Company,
ch., is made in sheets measuring 10½ in from center to center of
and having four ribs 1 3/16 in in height, and also in sheets 14 in in
g three ribs. There is also a type known as the Deep Rib. The
5, 8, 10 and 12 ft. The sheets are of No. 24, 26 or 28 United States
are furnished either flat or in various types of curves. (See, also,

sh. Corr-mesh is furnished by the Corrugated Bar Company,
Y., which supplies, also, special clips for splicing and fastening the
ribs, ¾ in in height are spaced uniformly 3 5/32 in on centers, the
12⅝ in from center to center of outside ribs. The mesh is fur-
e standard United States gauge, Nos. 24, 26 and 28, but it can be
other gauges if required. Standard sheets are 6, 8, 10 and 12 ft
it may be had up to 14 ft. This metal is always painted before it
nless otherwise ordered.

Self-Centering. The Youngstown Iron Steel Company, Youngs-
manufactures the Duplex Self-Centering. It is 23 in in width, is
lengths of from 4 to 12 ft, and in No. 24, 26 and 28 metal, United
e. It weighs 1.37 lb per sq ft for the No. 24 gauge, 1.03 lb for the
e and 0.86 lb for the No. 28 gauge; and it has a corresponding cross-
a per foot of width, of 0.411, 0.308 and 0.257 sq in.

e. Keyridge reinforcement is made by the Cellular Metal Com-
. 24, 26 and 28-gauge, black steel, 24 in in width and in lengths up
t is furnished with or without perforations. The ridges are spaced
enters, each sheet containing 7½ ridges. The sheets are so placed
t half-ridge on each sheet overlaps the first full ridge on the adjoin-

economy of this system is being substantiated, and particular
in combination with a light structural-steel fire-proofed frame.

Waite's Concrete Beam. In Fig. 48 is shown a type of
CONSTRUCTION that has been used by the Standard Concrete

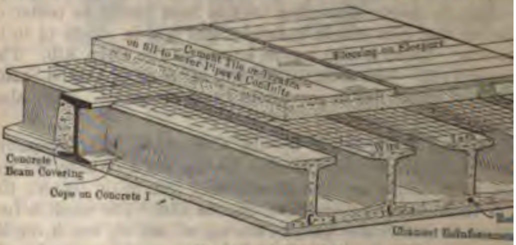
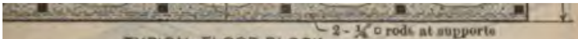


Fig. 48. Waite's Concrete I Beam

New York City in a number of buildings. The floor-constru
series of concrete **I** beams 10 or 12 in in depth, supported on th
the steel beams, which are spaced from 5 to 7 ft apart. The c
set about 12 in apart and the spaces between the lower flange
a cinder concrete of the same composition as the **I** beams.
concrete beams is placed a metal fabric of small mesh on whi
slab is laid. This makes a comparatively light floor-constru
the large spaces between the concrete beams The concrete
at the shop and allowed to harden before they are sent to
the lower flange is inserted, as shown, a steel reinforcement,
or other cross-section, to furnish the necessary tensile stren
are cast with the proper lengths, in accordance with the dr
slight variations at the building are made up by filling the sp



TYPICAL FLOOR BLOCK
6'0" Wide 13'0" long. Total Weight 4867# or 62.3# sq.ft.
Reinforcement 1 - ⅝" x ⅝" Havemeyer Bar in each Web
Mixture (1 - 2½ - 3½) top inch 1-5 Mortar

Fig. 49. Siegwart Reinforced-concrete Floor-construction

sfactory four-hour fire-test with a load of 150 lb per sq ft, followed e by a test with a load of 600 lb per sq ft. It is claimed for this t using the same working units for the strength of the material, the t of the construction is only one-half that of a monolithic reinforced- or designed to carry the same load with the same percentage of nt. "The Siegwart Company claim their method to be much n monolithic floors. From quotations furnished by their Canadian he price in Montreal is quite a little less than the author's expe- onolithic floors in the same city, ranging from 17 to 26 cts per sq ft, various spans and loads."* A modification of the Siegwart system veloped by Grosvenor Atterbury, and has been employed in two-story tory residence-buildings for the Sage Foundation Homes Company Hills (Long Island), N. Y.

nax Floor-System. This system (Fig. 50) was designed by S. M. and is marketed by the Climax Company, Chicago, Ill. The design that of the Siegwart floor-system.

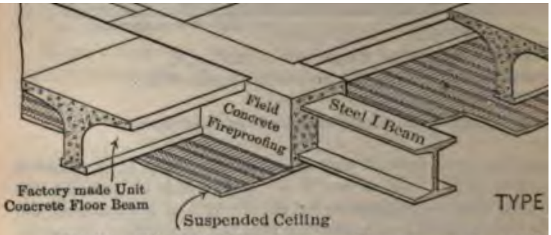
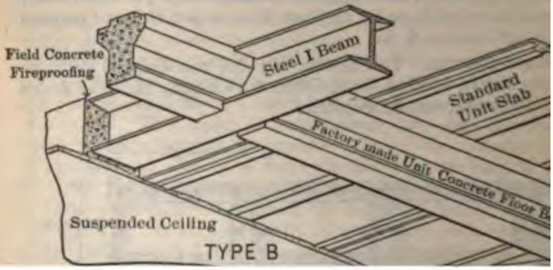
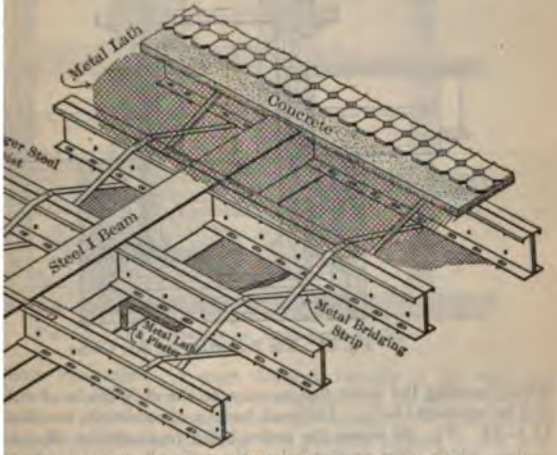

Field Concrete Fireproofing
Steel I Beam
Factory made Unit Concrete Floor Beam
Suspended Ceiling
TYPE

Fig. 51. Watson Reinforced-concrete Floor-construction. Wit

the Unit Construction Company of St. Louis, Mo. Beams cast with unit frames in horizontal molds and slabs are made forms. In the American School Board Journal for August, 191 Skinner describes the construction and erection of a story-and-


Field Concrete Fireproofing
Steel I Beam
Standard Unit Slab
Factory made Unit Concrete Floor B
Suspended Ceiling
TYPE B

;ing, to give additional rigidity. A typical floor-construction is 53. The steel floor-joists are covered above with a concrete slab expanded-metal lath, and the lower flanges are protected by a l lath and cement plaster attached to the joists by means of the metal-lumber joists frame into ordinary steel girders, resting on ıs shown in the drawing. The joists are cut to length at the rly marked and tagged and, with the erection-diagrams, are



53. Berger's Metal Lumber and Concrete Floor-construction

site. All joints and splices are riveted in the field. The steel be properly incased in some fire-proof covering. The materials construction are manufactured by the Berger Manufacturing Canton, Ohio, which publishes safe-load tables for metal- and channel-studs for spans of from 4 to 20 ft. This system, the use of steel joists and girders, not thoroughly incased with erials, should not be considered thoroughly fire-resistant under It has been extensively used to replace combustible building-specially in residence-buildings.

of Girders and Beams. No form of floor-construction can be roughly fire-proof unless it includes a protection of the lower teel beams and girders, or provides for the protection of all steel struction or support. The material used for the protective cover-ly the same as that used in the floor-construction itself. The erials are tile, either dense, porous, or semiporous, and concrete ers, stone, or slag. Plaster compositions have also been used, ommended. (See page 818.) Beam-protection, where the floor-ncases the sides of the beams, as in Figs. 17, 20, or 39, should han 1 in thick. Where paneled ceilings are used, that is, where of the beams is *below the lower side of the floor-construction, 35, or 44, the protection should be increased* to at least 1½ in

Tile Beam-Protection. When tile is used, there are two types
tion. In one case the blocks incasing the bottom flanges of the steel
at the middle of the lower side of the flanges; in the other, they
under the edges of the bottom flanges and hold flat tiles with
against the lower side of those flanges. The former is the better
in the latter the danger of breakage of the part extending under the
supplemented by the possibility of an omission of the flat protection-til

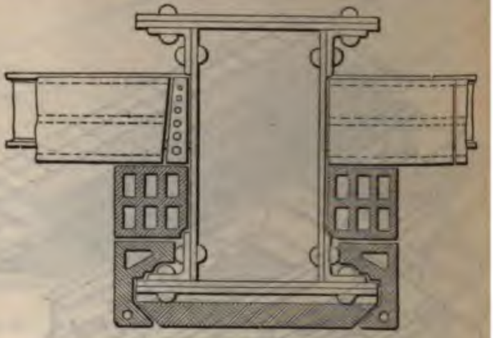


Fig. 54. Tile Protection for Box Girder

blocks incasing the lower flanges may be the skew-backs of the
may be separate blocks. Different forms and conditions are illustrat
15 to 27. Fig. 29 shows the entire beam protected by blocks on
Girders, which often project below the ceiling-line, are much more
the effects of fire and water than the floor-beams, and they should
the most efficient protection. As a rule, such girders should be

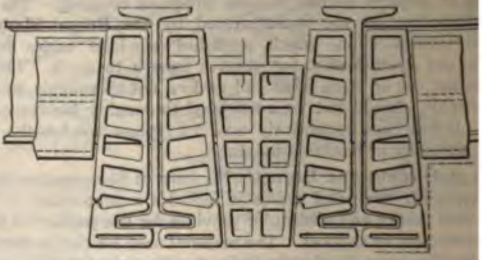


Fig. 55. Tile Protection for I-beam Girders

not less than 4 in of terra-cotta protection at the sides and 1½
on the lower side with a space of ¼ in between the terra-cotta
girder. Figs. 54 and 55 are typical methods of protecting girders
hollow tiles. The bottoms of the skew-backs (Fig. 54) are
spreading by wire ties placed in the end-joints between the
hooked into the round holes in the skew-backs. Single-beam

shown in Figs. 25 and 56, the latter figure showing more par- protection of a beam at the side of an opening in the floor.

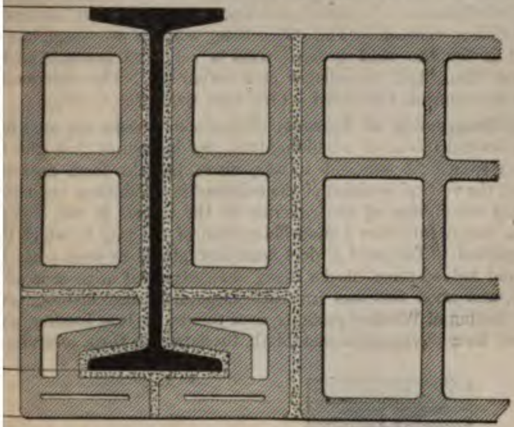


Fig. 56. Tile Protection for Single-beam Girder

Beam-Protection. A more thorough incasing of the webs and of beams and girders can be accomplished by the use of concrete. fire-proof character of cinder concrete makes it the best material pose. If of sufficient thickness and properly applied, it will hold

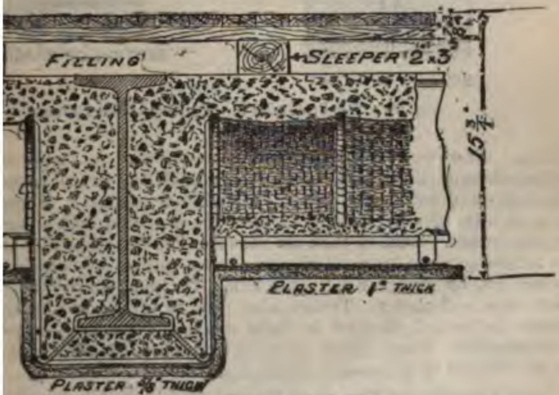
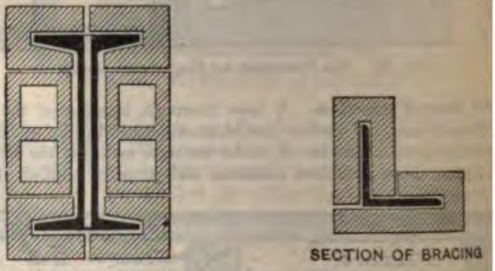


Fig. 57. Concrete Protection for I Beam

hout reinforcement, around the flanges of beams and girders is less than 2 in thick, wire or metal lath, wrapped around the be embedded in it. A common form of concrete-protection is 3. Sometimes the soffit of the beam is protected by a concrete

slab with an insulating air-space. This method is one that may b
tageously used for the protection of girders. A fire-test of this form o
protection made in the Butterick Building, New York City, thorough
lished its efficiency. A typical girder-protection of cinder concrete is
the Roebling method in Fig. 57. Hung ceilings are sometimes us
protection for the steel beams. This is very bad practice, as thes
are more than likely to collapse in a severe fire. The experience in
timore fire confirms this belief. (See, also, pages 780 to 782.)

The Fireproofing of Trusses. When steel trusses are used to sup
roof or several stories of a building it is very important that they be p
not only from heat sufficient to warp them, but also from expansion s
to affect the vertical position of the columns on which they are supporte
following description of the covering of the trusses in the Tremont
Boston, Mass., furnishes a good illustration of the way in which this s
accomplished: "The steel girders were first placed in terra-cotta block
sides and below, these blocks being then strapped with iron all aro
girders, and upon this was stretched expanded-metal lathing, covered
heavy coating of Windsor cement; over this comes iron furring, which
a second layer of expanded-metal lath, the latter, in turn, receiving the


SECTION OF STRUT

SECTION OF BRACING

Fig. 58. Tile Protection for Members of Steel Trusses

plaster. There is, consequently, in this arrangement for fire-protecti
a dead-air space, then a layer of terra-cotta, a Windsor cement covering,
dead-air space, and finally, the external Windsor cement." Numerou
of terra-cotta tiles are made for incasing the structural shapes commo
in steel trusses. Some of these are shown in Fig. 58. The tiles shoul
be secured in place by metal clamps passing entirely around the en
better still, by wrapping with wire lath. The tiling should then be
with hard wall-plaster. Trusses, also, may be fire-proofed by compl
casing the several members in cinder concrete, either with or witho
reinforcement. When trusses are to be fireproofed, the additional wei
be provided for in the strength of the truss.

Steel Framing for Fire-proof Floors. Before the framing-plans of
ing can be made, it is necessary to decide, in a general way, upon the
OF FLOOR-CONSTRUCTION or fireproofing that will be employed; thus, if
of the LONG-SPAN SYSTEMS, such as the Herculean, Johnson, and
the concrete systems, is to be adopted, the girders should be spac
the floor-construction will span between them, without floor-beam

vhile if an ORDINARY FLAT-TILE ARCH is to be used, floor-beams will
spaced from 5½ to 9 ft apart, and these beams must be supported
s indicated in Fig. 60. When there are no floor-beams, a STRUT-
l be riveted between the columns, as in Fig. 59, to hold the latter
ing erection and to stiffen the building. It should be remembered
or-beams spaced not more than 7 ft on centers, almost any system

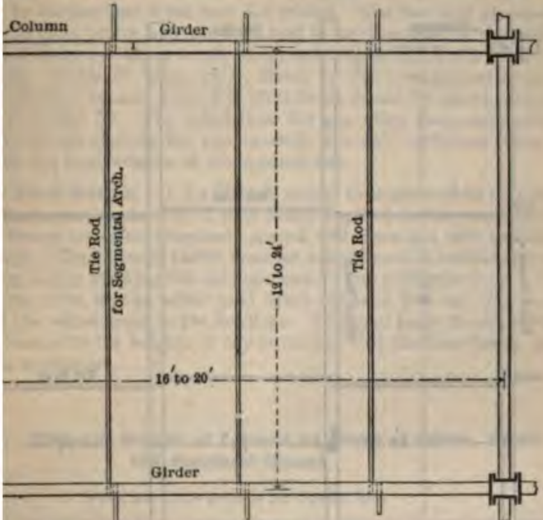


Fig. 59. Steel Floor-framing for Long-span Construction

truction may be employed; while if the floor-beams are omitted,
v systems to select from. With any form of filling between beams
ss steel is required for moderate than for excessive spans of beams

tions for the Steel Framing. The computations for the steel
irders of a fire-proof floor are very much the same as for a wooden
load or loads which any given beam is required to support are first
d then the beam of the necessary size to support the load is selected.
AD for any fire-proof floor may be estimated with sufficient accuracy
f the data given in this chapter in connection with the different
loor-construction. The dead load should include the weight of the
fireproofing, including all concrete filling, the plastering, furring,
ling-strips and flooring. The LIVE LOADS may be estimated by
e data given in Chapter XXI, pages 718 to 721.

The best arrangement for the columns in a retail store is to set
n centers in one direction and 19 ft 6 in in the other. It is decided
irders as shown by Fig. 60, and to put a *beam opposite each column*
ams between the columns. It is required to determine the proper
eams and girders, using an ordinary end-arch construction between

Solution. From Table VII, page 837,
which it is advisable to use is 10 in, but
beams it will be better to figure on a 1
on top. The weight of the 12-in arch wi
probably require 2 in of concrete filling
1½ in of light filling between nailing-strip

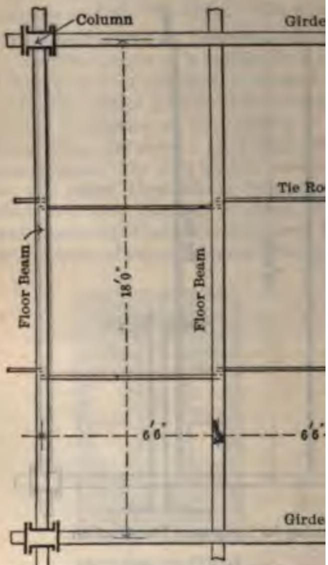


Fig. 60. Steel Floor-framing

flooring and nailing-strips will weigh abo
5 lb, and we must allow at least 6 lb per s
selves. These make a total dead weight
a retail store should be taken at 150 lb p
foot on the beams of 229 lb. The total lo
supporting will be 6½ ft by 18 ft by 229
assumed to be uniformly distributed. F
this load, with a span of 18 ft, will requir
42-lb beam. The latter will be both stron
thickness of the floor by 3 in and require

The girder must support two concentrat
On page 566 it is stated that when a be
at points one-third the length of the spa
formly distributed load may be found by
plying 26 793 lb by 2⅔ we have 71 448 lb
the girder, to which should be added the
a standard 24-in 80-lb beam (Table IV, p

ad of using tile arches between beams 6½ ft apart, we conclude to lerculean or Johnson construction spanning from girder to girder, we ıme our floor as in Fig. 59. For this span we should require 10-in tiles, 55 lb per sq ft. Allowing 8 lb for 1 in of concrete, 9 lb for filling, 4 lb g and strips and 5 lb for plastering, we have 81 lb as the dead load per ɔt. We have added nothing for the weight of the girder, as this will fset by the portions of the floor not loaded. The live load per square be 150 lb as before, and the total load to be supported by the girder, 9 ft 6 in by 231 lb, or 81 081 lb, or 40.54 tons, which will require a ɔ beam (Table IV, page 577). Hence by this arrangement we save t of the floor-beams; but a 6-in strut-beam should be placed between ns, as in Fig. 59. The calculations for any other floor-construction r to the calculations for this example, the only variations being in ıg of the dead weights of the construction.

for Floor-Beams. It is a difficult matter to prepare tables that may lly used, showing the size of steel beams required for fire-proof floors, ıch beams are often irregularly spaced, and there is a wide variation l loads. The following tables, however, may be used in making approxmates and in checking the computations for any particular floor. The beams given may be safely used where the total live and dead loads :eed the values given in the headings. The total loads should include allowance for the weights of any partitions that the floor-beams may upon to support.

XIV. Sizes and Weights of I Beams for Floors of Offices, Hotels and Apartment-Houses

Total load, 120 pounds per square foot

Distance between centers of beams in feet									
4½		5		5½		6		7	
in	lb	in	lb	in	lb	in	lb	in	lb
6	12¼	6	12¼	6	12¼	6	12¼	7	15
6	12¼	6	12¼	7	15	7	15	7	15
6	12¼	7	15	7	15	7	15	8	18
7	15	7	15	7	15	8	18	8	18
7	15	8	18	8	18	8	18	9	21
8	18	8	18	8	18	9	21	9	21
8	18	9	21	9	21	9	21	10	25
9	21	9	21	9	21	10	25	10	25
9	21	9	21	10	25	10	25	12	31½
9	21	10	25	10	25	10	25	12	31½
10	25	10	25	12	31½	12	31½	12	31½
10	25	12	31½	12	31½	12	31½	12	31½
10	25	12	31½	12	31½	12	31½	15	42
12	31½	12	31½	12	31½	12	31½	15	42
12	31½	12	31½	12	31½	15	42	15	42
12	31½	12	31½	15	42	15	42	15	42

Table XV. Sizes and Weights of I Beams for Floors of Retail Stor and Assembly-Rooms

Total load, 200 pounds per square foot

Span of beams in feet	Distance between centers of beams in feet				
	4½	5	5½	6	7
	in lb	in lb	in lb	in lb	in
10	7 15	7 15	7 15	8 18	8
11	7 15	8 18	8 18	8 18	9
12	8 18	8 18	9 21	9 21	9
13	8 18	9 21	9 21	10 25	10
14	9 21	9 21	10 25	10 25	12
15	9 21	10 25	10 25	12 31½	12
16	10 25	10 25	12 31½	12 31½	12
17	10 25	12 31½	12 31½	12 31½	12
18	12 31½	12 31½	12 31½	12 40	12
19	12 31½	12 31½	12 40	12 40	15
20	12 31½	12 40	12 40	15 42	15

Table XVI. Sizes and Weights of I Beams for Floors of Warehous

Total load, 270 pounds per square foot

Span of beams in feet	Distance between centers of beams in feet				
	4½	5	5½	6	6½
	in lb	in lb	in lb	in lb	in
10	8 18	8 18	8 18	9 21	9
11	8 18	9 21	9 21	9 21	10
12	9 21	9 21	10 25	10 25	10
13	10 25	10 25	10 25	12 31½	12
14	10 25	12 31½	12 31½	12 31½	12
15	12 31½	12 31½	12 31½	12 31½	12
16	12 31½	12 31½	12 31½	12 40	12
17	12 31½	12 40	12 40	12 40	15
18	12 40	12 40	15 42	15 42	15
19	12 40	15 42	15 42	15 42	15
20	15 42	15 42	15 42	15 45	15

Tie-rods. In all segmental arches and other types in which a th exerted against the beams, TIE-RODS must be provided to prevent the from being pushed apart, and especially to prevent the outer bays from ing. They should run from beam to beam from one end of the floor other. If the outer arches spring from an angle, as in Fig. 14, the tie this bay should be anchored into the walls with large plate-washer tie-rods should be located in the LINES OF THRUST of the arches, ordinarily below the half-depth of the beams, and in some cases near

ind for 9-ft spans, ⅞-in rods, 4 ft apart.
IZONTAL THRUST of an arch may be found by the following formula:

$$T = \frac{3\,wL^2}{2\,R}$$

= pressure or thrust in pounds per linear foot of arch;
= load on arch in pounds per square foot, uniformly distributed;
= span of arch in feet;
= rise of segmental arch, or effective rise of flat arch, in inches.

: of a segmental arch is measured from the springing-line to the soffit
at the middle. For flat hollow-tile arches, the effective rise may be
m the top of the beam-flange to the top of the tiles. As the tiles
ject from 1½ to 2 in below the bottom of the beams, the effective
from 2 to 2½ in less than the thickness of the arch. For the interior
a floor, *w* may be taken for the live load only, but for the exterior
hould include both the full dead and live loads. Having found the
he arch, the SPACING OF THE RODS of any particular size may be
ermined by dividing the safe load given for that size of rod in the
age 388, allowing 16 000 lb UNIT STRESS, by the thrust. The result
spacing in feet.

. What size of tie-rods and what spacing should be used for the
ruction described on page 868, in the preceding example?

. The depth of a tile arch is 12 in, the dead load 79 lb and the
ive load 150 lb. The span between the beams is 6½ ft. Then, for
r arches, $w = 150$ lb, $R = 12 - 2\frac{1}{2} = 9\frac{1}{2}$ in, $L = 6\frac{1}{2}$ ft and $T =$
42.25)/(2 × 9½) = 1 000 lb. The tensile strength of a ¾-in rod, not
16 000 lb per sq in, is, from Table II, page 388, 4 832 lb. Divid-

5. Fire-proof R

Flat Roofs. Flat roofs are constructe
that the beams and girders are set so a
drain the water. As the ROOF-LOADS are
as there are no partitions to be supporte
considerably lighter than the floor-panel
tically the same for both. When the
the beams should be set so that the con
to the roof, and should have a nearly
amount of concrete required, and also th
with tin or copper, nailing-strips should
wooden floors, and the entire roof sheatl
laid over terra-cotta or concrete will ru
roofs may be built without woodwork
concrete is used for the roof-panels, the
and girders should be efficiently protecte
metal in the roof-space well protected.
are stair-wells or elevator-wells, the roof
severely tested by heat, in case of fire, th
ence has shown that this part of the bui

Pitched Roofs. Pitched roofs may
ing to the material that is to be used an

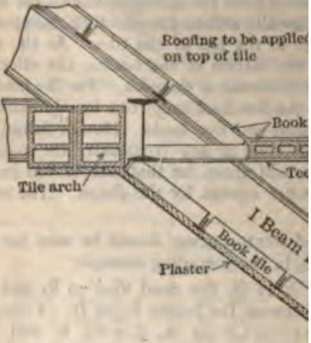


Fig. 61. Tile Fireproofin

ployed. When terra-cotta is to be used
method of construction is that which inv
beam rafters and T-iron purlins, set hor
than the lengths of the tile. Between
placed as in Fig. 61, and the roofing is ap
If the roofing is to be of slate or of clay
should be used between the tees, as the

* V

tiles. The same construction may be used for flat roofs; but on e expense of the tees it will usually be more expensive than the bove described, and not as strong or desirable. With the con- vn in Fig. 61, it is impossible, by any economical method, to ect the bottom of the T irons from the effects of heat.

inder concrete, or reinforced porous terra-cotta tile, Johnson s an excellent and also an economical construction for fire-proof Either of these constructions may be filled between or on top

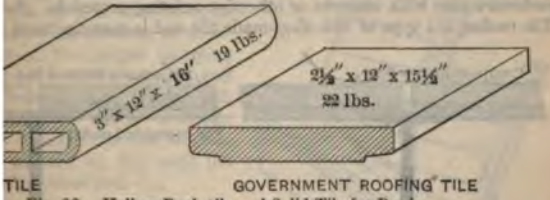
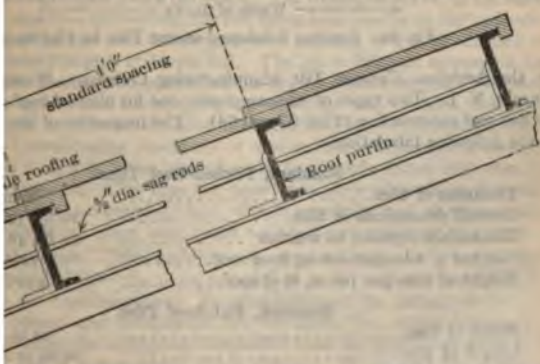


Fig. 62. Hollow Book-tile and Solid Tile for Roofs

without the use of purlins, except about once in from 6 to 10 ft, ing and to stiffen the roof.

plates of concrete, with expanded metal embedded, have been ed in spans of from 6 to 7 ft and in some cases even in 8-ft spans. is deposited on wooden centerings, as in the floor-construction, side is smoothed off during the setting and floated smooth and eive the roof-covering."*

vering, usually slate, or clay tiles, may be nailed directly to the der concrete holds the nails nearly as well as does wood. This



g. 63. Bonanza Reinforced-cement Tiles for Pitched Roofs

cinder concrete, as it is quite impossible to nail into rock concrete rete. In concrete roofs the rafters, also, should be surrounded held in place by metal lath. With terra-cotta roofs, the beams sed with terra-cotta blocks. *Fig. 62 shows the standard shapes nd solid roofing-tiles. These are made 2, 2½ and 3* in thick, and from, 16 to 24 in long. Three-inch book-tiles weigh about 13 lb per sq 2¼-in solid tiles about 16 lb per sq ft. Tiles of both of these shapes used for ceilings and where a light, fire-proof filling is required.

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* *Freitag.*

Reinforced-Cement Tiles. Cement tiles of interlocking types, made factory and reinforced with metal fabric or mesh, may be laid without directly on steel purlins. This type of construction, however, is suitable as a semifire-resisting roof-covering, as it is usually made with plates of cient thickness and does not contemplate the thorough incasing of the understructure with concrete or other fire-resisting materials. Bonanza Tile roofing is a type of this shop-made tile and is manufactured and

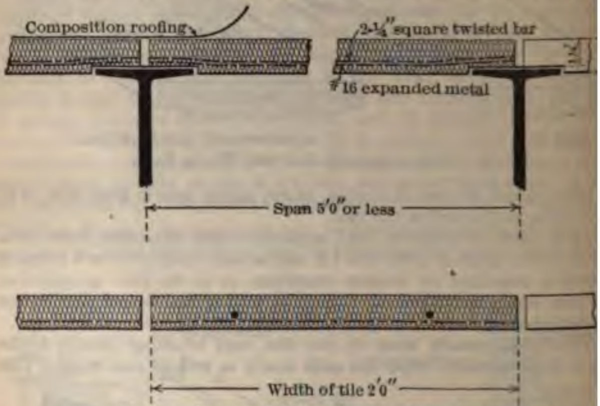

Fig. 64. Bonanza Reinforced-cement Tiles for Flat Roofs

by the American Cement Tile Manufacturing Company, Wampum, Lincoln, N. J. Two types of tiles are made, one for pitched-roof and th for flat-roof construction (Figs. 63 and 64). The properties of the tiles in the following tabulation:

Standard, Pitched-Roof Tiles

Thickness of tiles	⅞ in
Over-all dimensions of tiles	26 by 52 in
Tile-surface exposed to weather	24 by 48 in
Number of tiles per 100 sq ft of roof	12½
Weight of tiles per 100 sq ft of roof	1 330 lb

Standard, Flat-Roof Tiles

Width of tiles	24 in
Length of tiles	60 in or less
Thickness of tiles	1½ in
Reinforcement	No. 16 expanded metal and 2¼-in square, twisted ba
Weight of tiles	16 lb per sq ft

The flat-roof tiles are designed for and have been used in connecti buildings for manufacturing-plants on spans of 5 ft between purlin these spans they have been tested up to an ultimate live load of 250 lb p

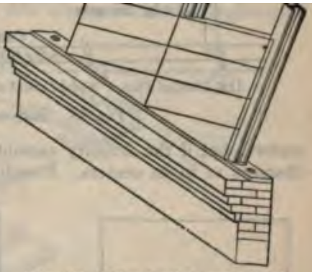

Fig. 65. Tiles for Mansard Roof

e the slates or tiles, and
etween the strips with
. This gives a better
roofing, and the wooden
affected by fire until the
lly destroyed.

ngs. The materials or-
for the roof-covering of
lings are: (1) tar and
halt and gravel or sand;
les, bricks or slate tiles
lt. Tar and gravel, or
and gravel, or sand, offer
of suitable for a fire-proof
hen a good quality of felt and distilled pitch or the best grades
used, make a very satisfactory covering. Such roofs, however,
newed about every ten years. The roofing is put on in the same
wooden construction, the felt being laid directly on the concrete.
st flat roof that can be put on a building is one of vitrified or
ver five plys of tarred felt. The felt is laid and mopped as for
d the tiles are bedded on the felt in cement mortar. Vitrified
square and 1½ in thick, are made for this purpose, and slate
are by 1 in thick, have been used. Flat, vitrified-brick tiles,
Gravel roofing should not be used on roofs which have an inclina-
¼ in in 1 ft. For pitched or inclined roofs, slates, clay tiles, or
be used. Clay tiles are superior to slate when exposed to fire
y to be preferred to slate; this is especially true of some of the
ing tiles. (See, also, pages 1496 to 1502, and 1509 to 1513.)

Ceilings. Office-buildings, apartment-houses, etc., having flat
eilings below the roofs in order to make a proper finish in the
for heat-insulation. In office-buildings the ceilings of the top
framed and constructed like the floors, but with a lighter con-

made by lacing wire, or by bending the iron. Where stiffened, wire as the Roebling or Clinton lath, is used, the channels may be spaced

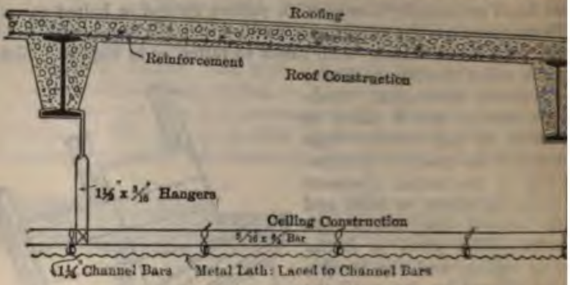

Fig. 66. Suspended-ceiling Construction

centers; but if the ordinary expanded laths are used, it is better to channels 12 in on centers. If ordinary lime mortar is used for pla 12-in spacing necessary.

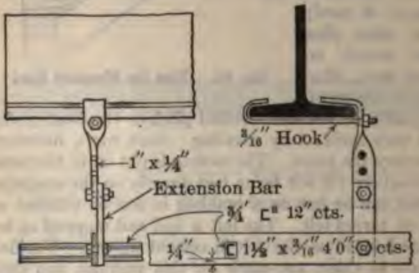

Fig. 67. Suspended Ceiling. Details of Two-bar System

Another syst which uses only horizontal bars, spaced close receive the lat which are sup hangers. With wire lathing, spaced not over and short han may be the ch tem; but witho ened lathing, t stiffness to the right-angles to the bars. Where the hangers are 3, 4 or 5 ft lon spans between the beams wider than 5 ft, the two-bar system, sho 66, requires less steel, for the reason that the channels, having spans of only 4 ft, may be made very light, and only one-third or one-fourth the number of hangers are required. In place of the small channels, small T bars or flat bars may be used, but when the bars are held by lacing, channels are preferable.

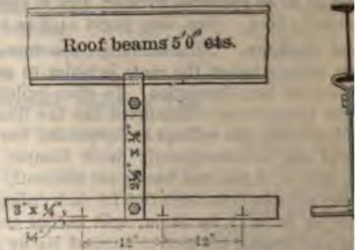

Fig. 68. Suspended Ceiling. Details of Tw

Figs. 67 * and 68 * show very satisfactory details for the constru

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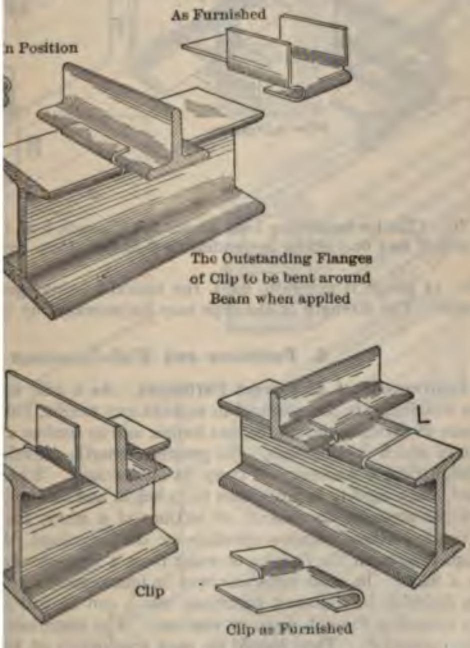
* From Fire Prevention and Fire Protection, J. K. Freitag, pages 687

nstead of the hook shown in Fig. 67, the hanger may be
e-half bending around one side of the beam-flange and the
he other side. Where the ceiling is suspended below terra-
-bolts are used for the support of the hangers. The ends
pporting the lathing are usually spliced by means of sheet-
5 in long, wrapped closely around the bars and hammered
ed ceilings under segmental or paneled floor-construction,
re employed, except that the hangers are replaced by clips
ars close to the soffits of the beams.

'astening Angles or T Bars to I Beams and Channels.
. A. Streeter patented a steel clip for connecting angles and


As Furnished
n Position
The Outstanding Flanges of Clip to be bent around Beam when applied
L
Clip
Clip as Furnished

s for Fastening Tees and Angles on I Beams and Channels

rithout drilling or bolting, and they have been extensively
roof-construction and suspended ceilings. Besides effect-
g away with the drilling and bolting required by the old
nable the workmen to make the connections in less time.
n easy method of adjusting T bars to any width of tile.
s with their applications are illustrated in Figs. 69 and 70.
e made on the same principle. The safe loads which may
s like *N*, or *NN* (*Fig. 70*), *and 1½ in wide, are as follows:*

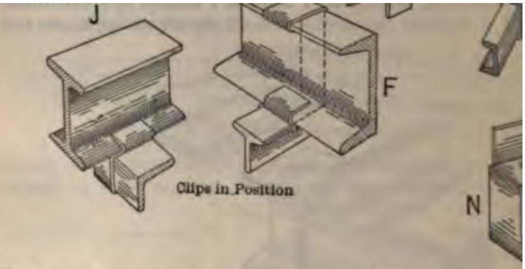


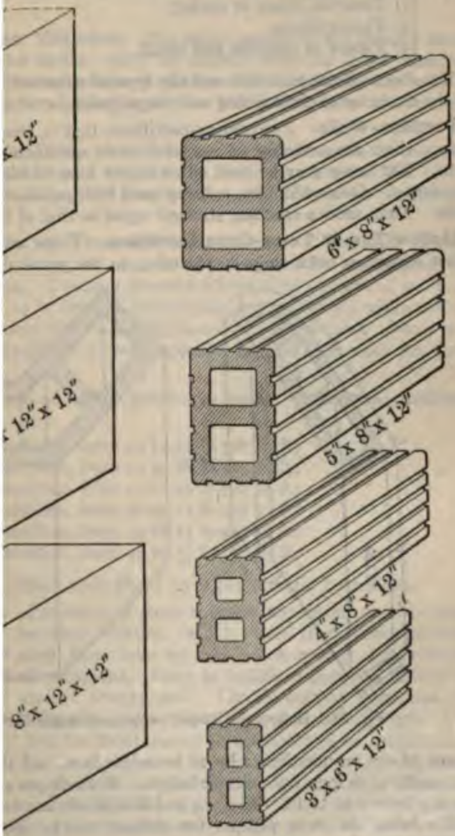
Fig. 70. Clips for Suspending Tees, Angles and Channels from I B
Clip *N* may be Used for Suspending Steel Shapes of any Sectio

No. 14 gauge is generally used, the material being specia
purpose. The strength of the clips may be increased by incr

6. Partitions and Wall-Coverings

Requirement of Fire-proof Partitions. As a rule the
proof buildings are not required to support any weight, but m
purpose of dividing the spaces into rooms, and to confine a fir
ment in which it originates. No greater strength, therefore
partition than is necessary to carry its own weight. Rigidit
quired, and a rigidity in proportion to its height and unsupport
partitions separate apartments or sections of a story, that
practically without window-openings or door-openings, the
enough to prevent the passage of water from a hose-stream
sage of flame. In other cases this may be unnecessary; in fac
seem desirable to construct partitions which can be easily r
a fire spreading through doors or windows. The materials of
be incombustible. They should be poor conductors of heat

t have not met the required fire-tests.* The standard
ociety for Testing Materials is based on the New York
sts require that the partition shall resist for one hour
f a wood fire, the heat of which has been gradually in-
ing the first half-hour and maintained at that tempera-



1. Hollow-tile or Terra-cotta Partition-blocks

f the time; and that it shall resist, also, for two and a
clusion of the fire-test, the application of a hose-stream

Bureau of Buildings, 1910 and 1911, for test-requirements and
tions.
Soc. for Test. *Mats.*

Types of Partitions. Fire-proof partitions that are in common use be grouped, according to the materials or the method of construction used, follows:

(1) Brick;
(2) Hollow tile or terra-cotta;
(3) Concrete, stone or cinder;
(4) Plaster-block;
(5) Plaster or concrete and metal.

The choice of the materials and the type of construction is largely by the character of the building and the purposes for which it is used.

Partition-Walls. For bearing-partitions, that is, those which support beams, there are probably no materials more satisfactory than brick and crete. The latter may be used either in the form of blocks, or may be into forms. Dense tile, also, is being used with satisfactory results for walls. Tests show a crushing strength equal to that of brick.

Hollow Tile or Terra-Cotta Partitions. These are usually built of either square or brick-shaped, according to the particular product used.

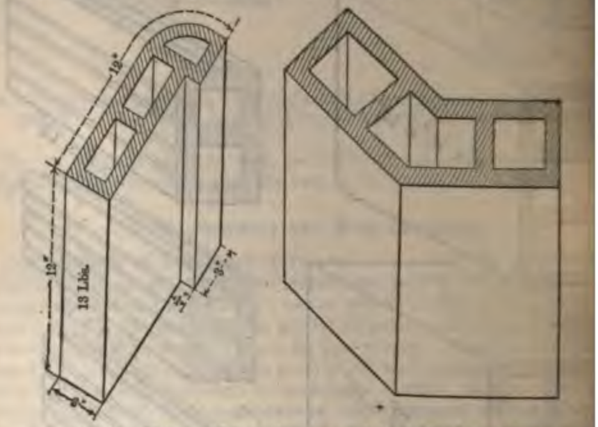


Fig. 72. Hollow-tile Round-corner and Angle Partition-blocks

square blocks are usually 12 by 12 in on the face, and the brick-shaped are usually 12 in long but vary in height. Both shapes are made in thick varying from 2 to 12 in. The 3, 4 and 6-in blocks are commonly used, the blocks being the most popular for ordinary work. For the more impor partitions, such as stair and elevator-enclosures, nothing narrower than 6-in blocks with the double row of cells should be used. The blocks are monly set with the voids horizontal, as in Fig. 71, the blocks breaking like bricks; but at the ends of partitions and in filling small spaces they sometimes set vertically. Fig. 71 shows typical shapes of both the square brick-shaped blocks. Fig. 72 shows round-cornered and angle-cornered tion-blocks, which must be set vertically.

"Terra-cotta partitions of a 2-in thickness have been placed on the ma but have not been extensively used. A 2-in terra-cotta partition of any str

: partition-blocks should be set in mortar made of one part parts cement and from two to three parts sand. The blocks et before setting and the partition wet down before the plaster-

Lengths of Terra-Cotta Partitions. "The safe HEIGHT artitions in inches may be approximated by multiplying the ies by 40. Common practice allows a safe height of 12 ft for -in and 20 ft for 6-in partitions. For partitions without side-NGTH should not materially exceed the safe height. Doors and ay be considered as side supports, provided the studs run from *

e WEIGHTS of either POROUS or DENSE terra-cotta partitions

2-in partition, from 10 to 14 lb per sq ft;
3-in partition, from 12 to 16 lb per sq ft;
4-in partition, from 13 to 19 lb per sq ft;
5-in partition, from 20 to 22 lb per sq ft;
6-in partition, from 22 to 23 lb per sq ft;
8-in partition, from 28 to 33 lb per sq ft;

istering, which adds about 10 lb per sq ft for both sides.

rtitions. Partitions of stone concrete are seldom used because ecessary for their erection, which make them comparatively ess reinforced they take up too much room. Furthermore aviest of all partitions. Even in buildings that are entirely of ete they are not always used. Cinder-concrete partitions are r and considerably cheaper than those of stone concrete. Yet oo heavy and too troublesome to construct to be satisfactory. titions tested and approved by the New York City Building hat consists of cinder-concrete blocks, 2½ and 3 in thick, the ng hollow, 12 in high and 18 in long. They have their edges

or suitable for first-class fire-proof bu
for these partitions is their great lig
with terra-cotta tiles. Plaster-blocks
a considerable holding-power for nai
the Bureau of Buildings, New York
able resistance to the flame and hav
any other form. They did not, how
of them being easily pierced, and all
by the water. An objectionable char
to absorb moisture while being store
when it is applied. This moisture w
where it is likely to injure the woode

These partitions are made in THICI
than 3 in in thickness generally bein
set with the cells horizontal. The ed
otherwise arranged so that the mort

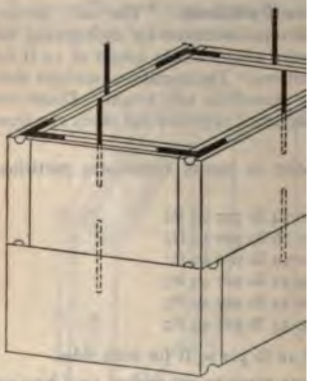


Fig. 73. Plaster-blocks. Doweled Con

partitions in the newer portions of
many other prominent buildings of
plaster-blocks.

Plaster-blocks make the lightest p
the blocks per square foot may be tak

Thickness of block, inches...	2
Weight in lb per sq ft.......	7

The plaster-boards, 1 in thick, wei
should be added to the weight of the
partition when plastered on both sides

Mackolite. A plaster block extens
made by the Mackolite Fireproofing
partition tiles are generally made in t
4, 6, 8 and 12-in thicknesses. The 3
others 30 in long, all the tiles being 1

---

* Patented by the Sanitary Fireproof

gitudinal strength to
ile the fiber makes
and elastic. The
in about half an
hich the blocks are
four days.

Partitions. What
e an economical fire-
l for partitions has
put on the market
sinite Company, of

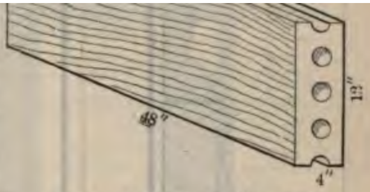


Fig. 74. Mackolite Partition-blocks

ty. The main feature of these partitions is the stud which is
rected in the same manner as a wooden stud in the ordinary non-
titions. The stud is composed of wooden nailing-strips com-
ted and embedded in a material known as Gypsinite concrete,
position and not a true concrete. The studs are carefully made
and true. Metal lath or plaster-boards are secured to the studs
completing the partition, which is about 4½ in thick. (Fig. 75.)
is slightly heavier than the ordinary partition of wooden con-
is quite as stiff and as strong as a good tile or other partition, and
ip feature of the studding facilitates the application of a wooden
id to be particularly sound-proof, and the spaces between the studs
ortunity to conceal pipes, wires, etc. Gypsinite studs in stock
by 12 in, and weigh 3 lb to the foot. They can be made any size
the partitions the studs are usually placed 16 in on centers and
ay be required. They are fastened to the floor or ceiling by the
d plates of the same material, or by light channel-irons, which are
fireproofing. The manufacturers believe that in large quantities
n be furnished as cheaply as wooden studs and that the partitions
l as cheaply as ordinary lath-and-plaster partitions.

ster-and-Metal Partitions. Thin partitions of plaster applied
and metal studs, made solid and finished about 2 in thick, have
ly used in fire-proof buildings. They are remarkably stiff, owing

pears to be subject to the CORROSIVE EFFECTS of the plaster. In the
of the Pabst Building, New York City, the metal lath used throug
partitions was found to be considerably corroded, after about four
though the lath had been painted. The subject is now being inve
the United States Bureau of Standards. The lath should in all ca

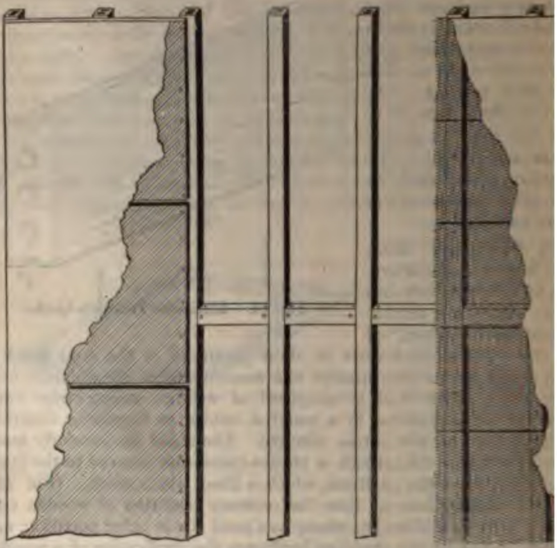


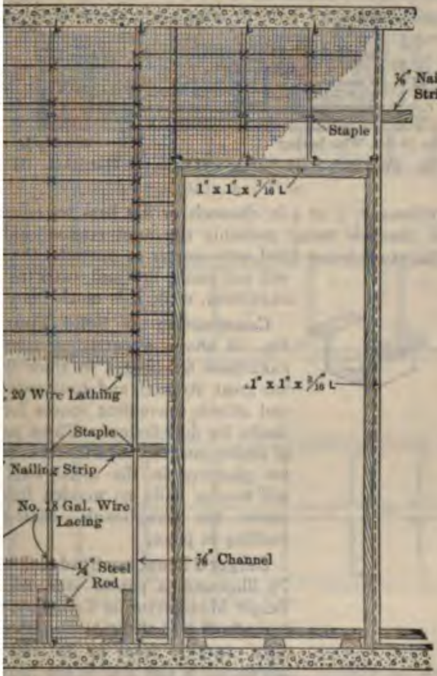
Fig. 75. Gypsinite Studs and Metal Lath and Plaster

tected against initial or incipient corrosion by painting or galvan
being embedded in the cementitious material.

Weight of Plaster-and-Metal Partitions. The WEIGHT of
partition, when dry, is about 20 lb per sq ft. The weight of partitio
thickness may be estimated on a basis of 120 lb per cu ft for plast
for cinder concrete, slightly tamped.

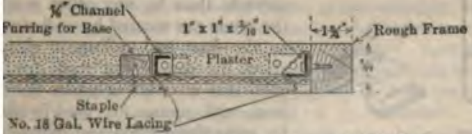
Construction of Solid Two-Inch Partitions. Figs. 76 and
usual method of constructing 2-in partitions. The studs, usuall
channels, are bent and punched at the ends, and at the bottom
wooden strips, which are first secured to the floor-panels, or to th
steel beams where the partitions come over them. These wooden
been found necessary as a sort of cushion to allow the studding
case of fire. At the top, the studs are nailed to the underside of the
or, if there is a suspended ceiling, they are wired to the bars su
ceiling. At the openings, 1 by 1 by 3/16-in angles are used, and th
every 16 in for No. 12 screws, used in attaching the rough wood
the angles. After the studding is in position, the metal lathing is
side of it with No. 18 galvanized wire. After the lathing is in p
penter should attach wooden grounds to secure the base, chair-

se grounds are secured by staples, and when the partition is very rigid. In plastering these partitions, five coats of d to make a good job; a scratch-coat on one side, a brown



. 76. Two-inch Solid Plaster Partition. Elevation

and the usual white coat on each side for finishing. It is in partitions that a HARD-SETTING mortar be used, such as g's Windsor Cement, Adamant, Rock Wall Plaster and many



Two-inch Solid Plaster Partition. Horizontal Section

tions acquire their STIFFNESS largely from the solidity of the he firmer and harder the plastering the more substantial the

Double Partitions. Electric wire
SOLID PARTITIONS; but if it is desire
that is, partitions with lathing on eac

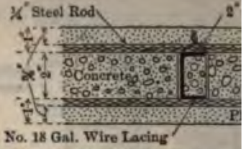


Fig. 78. Four-inch Solid Plaster an

these partitions, 2, 3 or 4-in channe
sheet-steel channels being probably
between the studs is not filled with
will not s
PARTITION

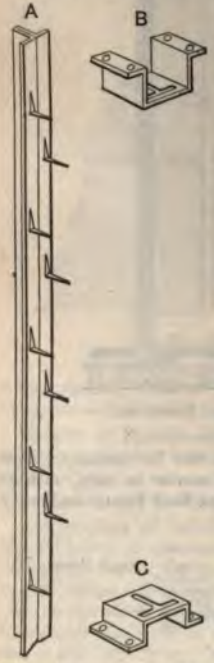
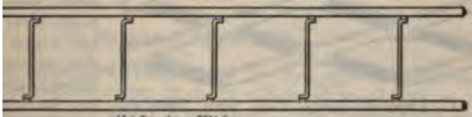


Fig. 79. Berger Studding or Furring and Stud-sockets

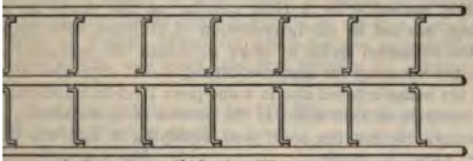
Constru
Fig. 78 s
PARTITION
has great
and afforc
jambs for
of cinder c
are plaste
will receiv
fasten th
molding ir

Berger'
79 illustra
Berger Ma
is made of
sizes, var
advantage
the lath.
both sides
right-angl
placed aga
the meshe
a hammer
securely t
the studs
to the flo
above the
partitions
irons with
By using t
saving in
shown in
supporting
being secured to the flanges of the be
strips and channels, also, are made o

2¼ Inches and 3¼ Inches Wide.



4¼ Inches Wide.



6¼ Inches and 8¼ Inches Wide.

Fig. 80. Rib-stud for Plaster Partitions

ths up to 18 ft. The studs are made of OPEN-HEARTH STEEL,
eighing 0.55 lb per ft and the three-rib studs, 0.85 lb per ft.
TITIONS with ¾ or 1-in channels or studs, the studs should
to 16 in on centers, depending upon the stiffness and rigidity
in spacing should never be exceeded when the height of story
. For HOLLOW PARTITIONS with 2-in studs, the studs can
centers for story-heights of 16 ft or less, when No. 24 (United
nded metal or No. 18 (United States gauge) wire lath, 2½ by
oyed. For greater heights the spacing should never exceed

that metal lath be always at least painted to prevent initial
the lath can be covered by the mortar. Galvanizing is necessary
is danger of moisture reaching the lath while it is without a protecti
lime or cement. Where a particular type of lath is not mentioned
fication, it should be generally described as follows: "painted or galv
24-gauge expanded-metal lath, weighing not less than 3½ lb per sq yd,
or galvanized woven-wire cloth, No. 19 gauge, 2½ meshes to the
stiffeners placed 8 in on centers and weighing not less than 3½ lb per

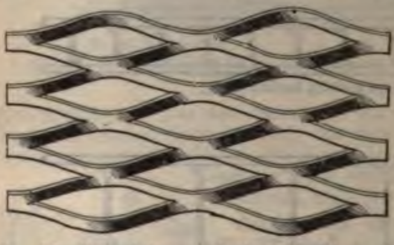


Fig. 81. Expanded-metal Lath with Diamond-shaped Mesh

Metal lath should be
that it will take pla
key it thoroughly a
embed itself in it.
characteristics of
metal and woven-
which make them
sheet lath. Sheet
economical in the
tar, which merely
side of the lath
through the perfora
out thoroughly emb
metal.

The difficulty of
plain wire lath tight enough to make a firm foundation for plaste
resulting necessity for a close spacing of the studs to secure th
bearing, has led to the introduction of stiffened wire cloth and ribb
rugated expanded metal in order to obtain the necessary rigidity.
come the necessity for separate bearing-studs, expanded-metal and
laths are manufactured also in a one-piece steel-lath-and-stud.

EXPANDED METALS differ in the process of manufacture. One typ
by simply slitting the sheet and deploying it into the diamond
other type is made from thin strips of soft, tough steel by a mechanical process which pushes out and expands the metal into the mesh and at the same time reverses the direction of the edge, so that the flat surface of the cut strand is nearly at right-angles to the general surface of the sheet. It is claimed that the
COLD WORKING of this low-carbon steel increases the ELASTIC LIMIT
MATE STRENGTH. In specifying expanded metal, it is necessary
weight of the finished product per square yard as well as th
metal, as the strands may be of various widths. EXPANDED ME
either with diamond-shaped (Fig. 81) or rectangular meshes (Fig.
laid with the long strands perpendicular to the studs, the lath
tangular mesh is the stronger of the two. Rigidity is also obtain
rugating and expanding the metal in various forms, which make t

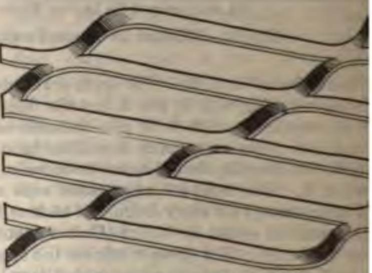


Fig. 82. Expanded-metal Lath with Rectang

[illegible]
vire lath;
ain,
iffened.
laths and their characteristics are given in the following para-

(1) Expanded-Metal Lath

piral Expanded Lath, made by the American Rolling Mill
ldletown, Ohio, comes in sheets 16¼ by 96 in, of Nos. 24 and 26
g respectively 4 and 3 lb per sq yd. This special-twist lath has
nsively in United States Government work.

mond-Mesh Lath. This lath is made by the Berger Manu-
pany, Canton, Ohio. It is furnished in sheets 18 by 96 in, of
and 24 gauge, weighing respectively 2⅓ lb, 2½ lb, 3 lb and 3.4 lb
t is made of Toncan Metal for which greater HOMOGENEITY is
for charcoal-iron and steel, and less liability to CORROSION or

ath. Bostwick lath is made by the Bostwick Steel Lath Com-
hio. It is furnished in sheets 14 by 96 in, approximately 1 sq yd,
Nos. 24 and 27 gauge.

Lath. This material is manufactured by the Consolidated
tal Company, Pittsburgh, Pa., and is furnished in two styles,
lcrete A and Steelcrete B lath, for exterior stucco-work, and in
n as Steelcrete Diamond lath, for interior work. The Steelcrete
e in Nos. 22 and 24 United States gauge, in sheets 18 by 96 in,
37 and 3.56 lb per sq yd. Steelcrete B lath is made in No. 27
ighs 2.41 lb per sq yd. The Steelcrete Diamond lath is made in
27 in wide and 96 in long, in Nos. 24 and 27 United States gauge,
ctively 3.57 and 2.48 lb per sq yd.

l-Mesh Lath is made by the Eastern Expanded Metal Com-
Mass., in the following sizes: No. 22 gauge, in sheets 20¼ by
4 lb per sq yd; No. 24 gauge, in sheets 22½ by 96 in and 30 by

Key Expanded-Metal Lath, made by the General Fireproofing
Youngstown, Ohio, is furnished in sheets 24 by 96 in, in Nos. 27,
gauge, weighing respectively 2.34, 2.50, 3.00 and 3.40 lb per sq yd.

Kno-Burn Lath, made by the North Western Expanded Metal
Chicago, Ill., is furnished in sheets 18 by 96 in, in Nos. 27, 26, 25
weighing respectively 2⅛, 2½, 3 and 3.4 lb per sq yd. When m
special ACID-RESISTING sheet steel, this lath is sold under the name X
Expanded Metal Lath.

An Expanded-Metal Lath is manufactured by the Roebling
Company, of New York City, in rolls which are to be applied par
furring, thus eliminating all transverse laps between the studs. It
rolls of 150 lin ft and in widths of 25 and 33 in, for furring set
on centers. The material is furnished in Nos. 24 and 26 United St
either galvanized or painted.

Ribbed or Corrugated Lath. Cleveland lath, made by the Gar
Steel Company, Niles, Ohio, is a corrugated diamond-mesh lath,
two grades, A and B, the former being the more suitable for stuc
lath is made in sheets 16½ by 96 in, of No. 27 and 24-gauge meta
respectively 3⅛ and 4 lb per sq yd; and the B lath, in sheets 18
Nos. 27 and 24-gauge metal weighing respectively 2⅝ and 3⅛ lb per sq
eighth pound per square yard should be added to the weights if the
vanized. This lath is reversible, the two sides being identical.

Herringbone Expanded-Metal Lath (Fig. 83), made by The Ge
proofing Company, Youngstown, Ohio, is furnished in two styles,

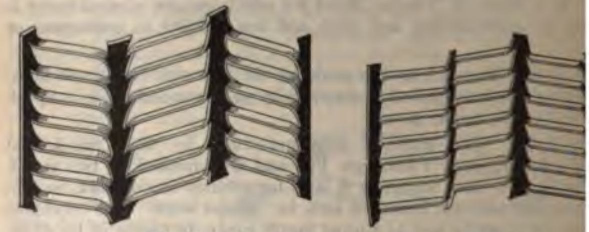


Fig. 83. Expanded-metal Lath, Herringbone Mesh

Style A is made in sheets 13½ by 96 in (1 sq yd), of No. 28-gauge m
ing 3 lb per sq yd. Style BB is made in sheets 20¼ by 96 in (1
Nos. 27, 26 and 24-gauge metal, weighing respectively 2¼, 2½ an
sq yd. It is made of American INGOT-IRON with RUST-RESISTIN
Ribs are set across studs and slope down towards them.

Sykes Expanded Cup-Lath, made by the Sykes Metal Lath
Company, Niles, Ohio, is furnished in sheets 18 by 96 in, with
coating, or painted black, or galvanized. It is made of Nos. 27,
gauge metal, weighing respectively 2.8, 3 and 3.7 lb per sq yd.

Standard Rib-Lath, made by the Trussed Concrete Steel Comp
Mich., is furnished in sheets 20¼ by 96 in, in grades 1, 2 and 3, wei
tively 2.74, 3.42 and 4.10 lb per sq yd. This company makes also
Plate Rib-Lath, which is about 35% heavier and more rigid, per
spacing of the studs.

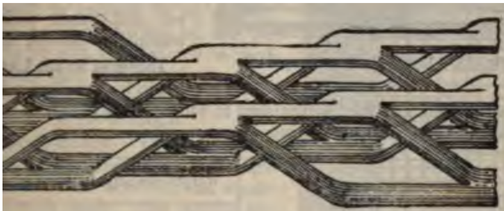


Fig. 84. Truss Metal Lath

Nos. 26 and 28-gauge metal, weighing respectively 80 and 66.7 lb A partition constructed of this lath in one of the test-structures niversity, New York City, passed through and withstood, with- distress, the fire and hose-streams of five successive tests.

ing, made by the General Fireproofing Company, Youngstown, ied in sheets 28 in wide and of any desired length up to 14 ft, and 24-gauge metal, weighing respectively 0.60, 0.72 and 0.96 lb lath may also be obtained in special gauges from Nos. 24 to The width of 28 in means the covering capacity, as laps are pro- tside ribs. This company makes, also, a lath known as Trussit, in wide and in lengths up to 12 ft, of Nos. 27, 26 and 24-gauge g respectively 0.71, 0.77 and 1.02 lb per sq ft. (See, also,

de by the Trussed Concrete Steel Company, Detroit, Mich., three types known as 4-Rib, 3-Rib and Deep Rib. The first is wide and the others in sheets 14 in wide. (See, also, page 859.) furnished in Nos. 24, 26 and 28-gauge metal. The standard , 10 and 12 ft. Other lengths below 12 ft are cut, but the waste f the purchaser. Hy-Rib sheets interlock at the sides and ends. allowance need be made for side laps, but for end-laps 2 in should aps over supports, or 8 in between supports.

Genfire Sheet-Steel Lath, made by the General Fireproofing Youngstown, Ohio, is furnished in sheets 13½ by 96 in (1 sq 24 by 96 in, weighing yd, painted unless other fied.

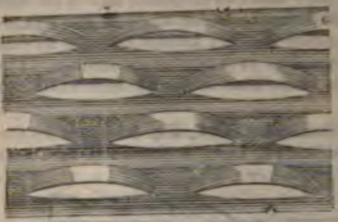


Fig. 85. Bostwick Truss-loop Lath

Sykes Trough Sheet L by the Sykes Metal Roofing Company, Nile furnished in sheets 13½ sq yd), 15½ by 96, 18 23½ by 96 in, weighing yd, and made with coating, or painted or ga

Integral Sheet Lath. made by the Berger Ma Company, Canton, Ohio, is furnished in widths of 24 in and in stock 4, 5, 6, 8, 10 and 12 ft, as follows:

Gauge	Weight per square yard in poun		
	1½-in rib	¾-in rib	1-in r
27	73	78	83
28	81	86	92
26	88	94	100
24	117	125	133

(3) Woven-Wire Lath

Woven-Wire Lath is furnished with or without STIFFENERS, either rods or V-SHAPED RIBS running through the wire mesh to re stiffen it. It is supplied painted or unpainted, or it is galvanized aft It can be furnished to order in any required width up to 10 ft. In than 18 in, there is a small charge for STRIPPING. Before ordering important to ascertain the proper width, especially in stiffened la desirable to have the edges of the lath lap at the supports where it iron furring. When the lath is not of the proper width the results are and there is liable to be a waste of material. The standard width of V-RIB STIFFENED LATH is 36 in. When beams or studs are spaced center to center, the lath should be 32 or 48 in wide.

The Clinton Stiffened Lath has corrugated-steel FURRING tached every 8 in, crosswise of the fabric, by means of METAL C strips constitute the FURRING and the lath is applied directly to th of the floor-joists, or to planking, furring, brick walls, etc. This lat 36-in widths, with 2½ meshes to the inch, and comes in 100-ft manufacturers of this lath make, also, a lath STIFFENED WITH R ⅛ to 1 in in diameter, spaced from 8 to 12 in apart. It can be ha vanized or japanned, and in thicknesses from 18 to 21 gauge. C WIRE LATH is furnished in rolls 200 ft long.

The Roebling Standard Wire Lath is made of plain WIRE CLO at intervals of 7½ in, STIFFENING RIBS are woven. These ribs hav section and are made of No. 24 sheet steel, ½ and 1 in in depth rib is the standard size for lathing on woodwork. This lathing

pplied directly to woodwork or to walls with steel nails driven
tom of the V, as shown in Fig. 86. The No. 20 V-rib stiffened
satisfactory surface for plastering, when attached to studs or
6 in apart. The 1-in V-rib lathing is used for furring exterior
ides an air-space between the wall and plaster. Where this
lied to light iron furring a 5/16 or ¼-in solid steel rod is substi-
rib, and the lathing is attached to light iron furring with lacing
is distinguished from the others by the term Solid-Rib Stiffened
e Roebling lath, whether plain or stiffened, is made with 2 by
nd 2½ by 4-in mesh, the last
nown as CLOSE WARP. The
h is adapted to all plasters
usual proportion of hair or
by 4-in mesh should be used
rs and thin partitions. The
nished in widths up to 10 ft,
ing 50 yd in length.

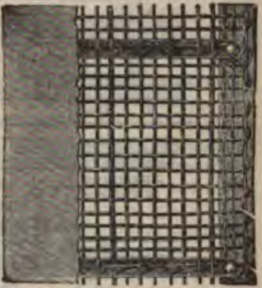


Fig. 86. Roebling V-rib Stiffened Wire Lath

all-Board. This is a com-
three layers of pure gypsum
ayers of wool-felt, the whole
½ in thick. The boards are
size, can be nailed to wooden
flat against solid beams or
n be cut with a saw. They
tage of being very light and
t little plastering material,
t reduction in the amount of water used in plastering. When
e amount of plastering is taken into account, this plaster-board
metal lath and but a trifle more than wooden lath with three

For plastering, the best results are obtained by applying first
hard wall-plaster, ¼ to ⅜ in thick, and when this is thoroughly
with a thin coat of regular hard finish of lime-putty and plaster.
tigations at the Underwriters' Laboratories "have shown Sacket
on Brand, to be suitable as a base for fibered gypsum plasters,
hed to walls, ceilings and partitions and coated with ½ in of
re-retarding properties considerably higher than those of wooden
n or lime-and-cement plaster." The Perfection Brand, Sackett's
⅜ in thick, and is attached with No. 10½, ½-in, flat-headed,
ails, 1¼ in long, and spaced not more than 6 in at each support.

laster-Board is composed of alternate layers of strong absorbent
l with fine annealed wire about 2 in on centers, and stiffened
h ½-in iron bands, No. 32 gauge, placed 8 in on centers. The
le up to a total thickness of about 1/16 in, impregnated with a
, and provided every 2 in with 3/16-in circular holes to key the
s added to the adhesive effect of the absorbent paper. It is
s 85 ft long and 34 in wide, nailed directly to the studs or beams
on centers, and lapped 2 in at all joints. This board is recom-
with hard-plaster mortars only, and forms a satisfactory basis
vork, in which the lap-joint obviates the cracking frequently
ordinary plaster-board construction.

uction. The most important partitions in a building are those
r shafts. Vertical openings through buildings form flues and
In all buildings, fire-proof as well as non-fire-proof, therefore,

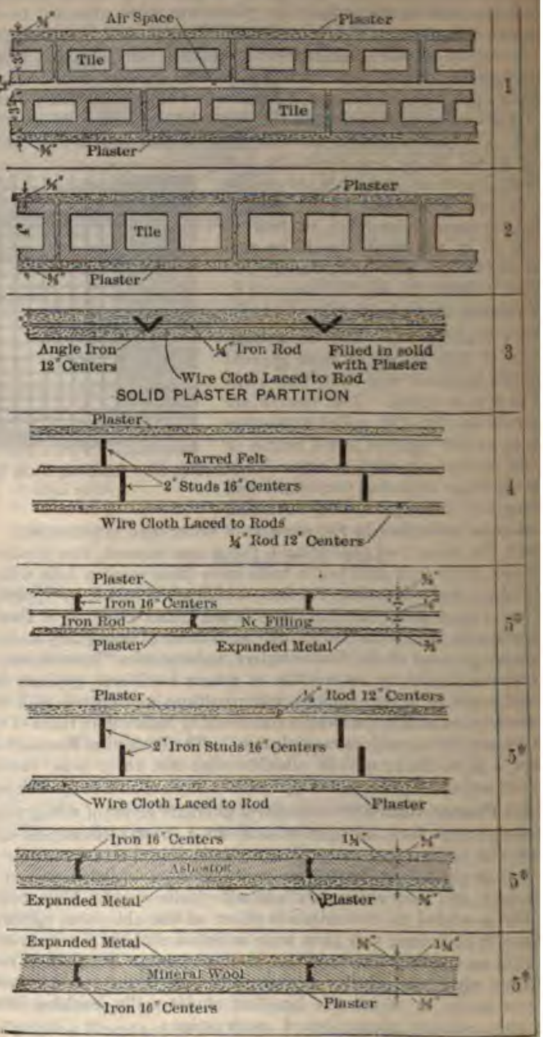


Fig. 87. Relative Deadening Properties of Partitions

s provided in such doors, although this is not good practice; if
d, wire-glass, only, should be used, in accordance with the limita-
n page 908. Open grille-work for passenger-elevator enclosures is
superseded by construction which is more fire-resisting. The
features of open grilles may still be retained for the fronts and doors
tors by using them in conjunction with approved wire-glass con-
n interior light-shafts and vent-shafts, openings must necessarily be
t here again the construction of the window-frames, sashes and
ld be as far as possible as described on pages 906 to 908. When-
upancy of a building admits, the stairs, also, should be inclosed in
ls, with fire-proof doors at the openings. Unless so inclosed the
rm flues for the flames, and the stairs themselves, consequently,
o intense heat. In such situations, even absolutely fire-proof stairs
used during a fire, and possibly it is for this reason that greater
ot been taken to make them fire-proof. Shaft-walls should in all
ied 3 ft or more above the roof.

g Properties of Partitions. The resistance to the passage of
gh fire-proof partitions is an important consideration in buildings
ng-apartments; and where the rooms are to be used as music-
comes a matter of still greater importance. In January, 1895, some
ade to determine the RELATIVE DEADENING PROPERTIES of the dif-
ons shown in Fig. 87, the object being to decide upon the construc-
ould be used in Steinway Hall, Chicago, Ill. The rank of the
titions tested, IN SOUND-PROOF EFFICIENCY, is shown by the num-
right of the partition-diagrams. The 4-in porous partition was
s not a success. In the Fine Arts building, in the same city,
tions, similar to No. 1, were used, and it is said that they were a
s. It is surprising to note that in the tests above mentioned, the
ster partition, No. 3, plastered with common mortar, ranked higher
ADENING PROPERTIES than those with double studs. The relative
including the plastering, was stated by the Illinois

Table XVII. Sound-Dead

No.	Room	Side	Scale	
1	E	Left	100	Cabot's
2	E	Right	95	Cabot's
3	E	Rear	95	Cabot's
4	C	Rear	85	Sackett
5	C	Left	85	Sackett
6	C	Right	80	Sackett
7	D	Rear	75	Metal la
8	D	Right	75	Metal la
9	B	Right	60	Two 2-i
10	A	Rear	50	4-in Na
11	B	Rear	50	3-in Ke
12	A	Right	45	3-in Na
13	B	Left	40	2-in Ke
14	A	Left	40	2-in Na
15	D	Left	30	2-in me

"Nothing more is to be inferred
'scale,' than that the first partition is

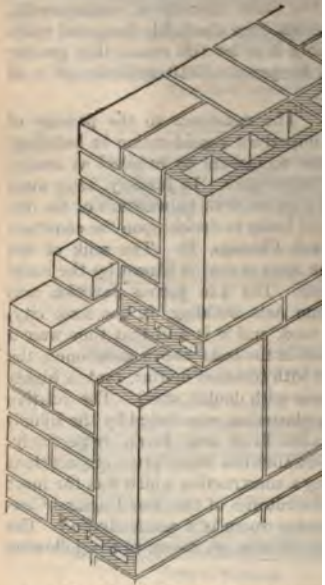


Fig. 88. Hollow-tile Furring-bloc

passage of moisture. This furring sho
never of wood. For this purpose fur

e required for bases, picture-moldings, etc. In some cities, e laws require that when furring-bricks or tiles are used unted as part of the thickness of the wall. Wire lathing, bs woven in every 7½ in, makes a good furring for brick applied and leaves air-spaces between the wall and plaster. ılso protect the walls from being warped by heat during a passage of heat through the walls in summer and winter.

To produce architectural forms in the interior decoration gs, METAL FURRING and METAL LATH are now almost uni- furring is always of a sham nature, and never employed y magnitude; so that the only requirement is that it shall d furnish a satisfactory ground for attaching the metal lath, s, false beams, etc., the furring-members are made of ees or channels, attached to the walls by means of nails, lts, and to the steel beams by means of bolts, hangers, clips, eces are bent or shaped to the approximate outlines of the k, so that when the lathing is applied it will require not n of plaster to give the desired outline. For plane surfaces, e brought to within ¾ in of the plaster-line. Deep beams, d by diagonal rods, to prevent distortion. All structural- d always be fire-proofed back of the furring. The lathing is g by means of No. 18 galvanized lacing wire. The spacing l be either 12 or 16 in, according to the kind of lath that is chases in walls are covered over, the covering should be rring and lath. The casings for vertical pipe-lines, also, nstruction and the space about the pipes at the floor-level dly with fire-proof material, to cut off all connection be-

7. Fire-proof Flooring

ing. The floor-surfaces of most fire-proof buildings consist ng secured in the usual manner to nailing-strips embedded n the filling above it. It is sometimes advisable to use in- . The New York City building code requires that in all

flooring-material which could be spread, without joints, ov
and at the same time be elastic, wear well, withstand water, a
be too expensive. Various mixtures of magnesite, asbestos,
mixed with linseed-oil, and some binder like chloride of mag
put on the market under different names, all more or less m
ments above stated and being, also, fire-proof. These mater
the form of a dry powder to the place where they are to be u
mixed with a specially prepared liquid. The resultant is
which is laid upon the surface to be covered in much the sa
nary cement or plaster is put on. The materials harden in f
in moderately dry weather, when the floor is ready for use
laid the floor presents a smooth, fine-grained and continuo
bling linoleum. These materials are made in various colors,
yellow, brown, gray, black, blue and green, and can be lai
concrete, asphalt, cement, or metals. Another advantage is
carried up on the walls so as to form a coved base, withou
Among the manufacturers furnishing such floorings may be
ican Monolith Company, Milwaukee, Wis.; Asbestolith Ma
pany, New York City; Robert A. Keasbey Company
Flooring), New York City; Hydrolith Company, Brooklyn
Kompolite Company, New York City; Marbleoid Company
Minnesota Faience Stonewood Company, St. Paul, Minn.;
Company, Chicago, Ill.; Keasbey & Mattison Company (
Lumber), New York City; Ronald Taylor Company, New
Warren Bros. Company, Boston, Mass.

8. Interior Finish and Fittings

Interior Finish. In buildings in New York City in which
be of incombustible material, the interior finish, also, includin
jambs, window-frames, sashes, bases and trims, must be mad
materials. The same materials that are accepted for floorin
this interior finish also. Several of the largest buildings in

be of wood covered with metal or of wood treated by some
y the Board of Buildings to render the same fire-proof."
ttempts in the United States to fire-proof the interior trim
hose made in New York City, about the year 1880, in the
ed woodwork, by the firm of Campbell & Bantossell of that
me, also, there were introduced along with various processes
dwork, FIRE-PROOF PAINTS. Later FIRE-PROOF WOOD was
wood which has the resin and other inflammable components
nd the fiber left. In the course of a few years the METAL-
ustry developed to such a stage that it was possible to trim
e interior of a building and keep a good appearance. No-
the Manhattan Life Insurance Company's Building and the
d, of more recent date, the Metropolitan Tower,* the Fifth
ling, the Germania Life Insurance Company's Building and
el, all in New York City; the Hoge Building, Seattle, Wash.;
ds, Los Angeles, Cal.; the Rockefeller Annex, Cleveland,

ished appearance of the STANDARD TIN-CLAD DOOR set men
t for use in interior finish which would lend itself to more
The KALAMEIN IRON and other METAL-COVERED work
meantime improvements were constantly being made in
AL doors and trim, and from about the year 1903 HOLLOW
for this work came into use. Owing to its generally superior
to the splendid enamel surfaces which can be given it by
esses, this type of interior finish has found favor in the eyes
d owners of modern offices, mercantile and public buildings.

† KALAMEIN IRON is the trade name given to one of the
teel products which is covered with a thin alloy of tin and
same way that galvanized iron by galvanic immersion is
"The name CALAMINE (with Galmei of the Germans) is
d to be a corruption of Cadmia. Agricola says it is from
n allusion to the slender forms (stalactic) common in the
" ‡ The term KALAMEIN is often used incorrectly, by

metal is steel, copper or bronze, to distinguish metal-covered fr
METAL construction; but the term is obviously misleading and ca
confusion. In several instances architects have specified KALAMEI
expecting BRONZE METAL to be used in the covering, whereas the man
interpretation of the specification was that KALAMEIN IRON was inte

Metal-Covered Doors, Frames and Trim. The cores of METAL
doors and frames are built up of oak or white-pine strips dovetaile
lengthwise to the grain. In gluing up the strips into stiles and rail
of each strip is reversed in order to resist the tendency of the cor
The stiles and rails are mortised, tenoned and box-wedged and the
covered with asbestos paper or board and enclosed with sheet metal,
(which may be painted to match a wooden trim, or electroplated w
brass or bronze), or solid, sheet copper, brass or bronze. For doors
4 in in width and 8 ft in height, both sides are often made of continu
of metal, which have the panels pressed into them by hydraulic pr
are without seam or joint. The metal sheets of the two sides, in o
door,* are made to overlap in a depression on the edges of the do
secured in place by screws which pass through both face-sheets. Th
thickness of this door is 2⅞ in. When these doors are more than
width, each face is generally made of two sheets which meet ove
stile and lock together with a flush double-lock joint. This make
row of vertical panels.

Metal-Covered Window-Frames and Sashes. Window-frames
as well as door-frames and doors, are made of METAL-COVERED WOO
is the metal usually recommended and preferred although Kalamei
be substituted when a much cheaper construction is necessary. T
metal may be painted and will give fair service but it is not rec
Galvanized iron and copper, also, are used.

"Window-frames and sashes of KALAMINE or of sheet-metal over w
are principally used for windows or skylights where the only dan
contact is through flying sparks. They are NON-COMBUSTIBLE
FIRE-RESISTING. The lights are usually of plate glass, expecially if
trim is used simply to comply with the law in those cities where no
ble windows and doors, etc., are required in buildings of a certain
height above fixed limits. Previous mention has been made of the
as demonstrated in the burning of the Kohl building in San Francisc
value, even as a substandard protection, has been pointed out; but
fire-resistance, KALAMINE windows, especially, are an unknown q
the resistance offered by the lighter members, such as sash-rails, is q
The better examples of the work present pleasing workmanship an
some composition could be used for the body instead of wood, withou
chemical action harmful to the metal, a superior type of KALAMINE

ries from 1½ to 2⅛ in.

Metal Doors. The Dahlstrom patent SHEET-METAL DOOR* is two No. 20-gauge, steel-plates, one stile and one panel-face being m each of the sheets, which are connected by interlocking seams on des of the panels and make practically a double door. In constructnels they are first lined with a sheet of asbestos next to the steel on and the space between is filled with a layer of hair-felt paper, which esilient filling that is a non-conductor of heat. The stiles are left strips of cork are laid perpendicularly across the center of each to metallic ring. The panels are then attached to each other to form y planting on and welding in place properly formed cross-rails, at d bottom, and wherever else they may be desired; the moldings are the molded stiles at the sides. The top and bottom edges of the hen reinforced with channels and bars, and the doors made perfectly nd rigid. The fire-resistance of this construction is increased by rivets or screws pass through from one side of the door to the other osed parts. The transmission of heat is thus avoided. While the ing put together, provision is made for attaching the hardware. doors have been put together, they are sent to the finishing departe the steel is thoroughly cleaned from all rust, grease or other im- They are then given six or eight coatings of enamel, being baked after tion of each coat in large ovens which are heated to 300° F. After oat of varnish is put on, they are usually rubbed to an egg-shell, , equal in quality to any hardwood-finish, and more durable because The surfaces can be grained to imitate with wonderful exactness such as quartered oak, mahogany, Circassian walnut, etc. If the to receive glass panels they are provided with detachable moldings glass in place.

the Dahlstrom, HOLLOW METAL type are installed in the corridors ons of the Singer Building and tower† and the United States Express

asbestos lining in the stiles and rails to prevent reverberation
many buildings that have been equipped with hollow steel d
this company may be mentioned, in New York City, the Wo
ing, the Germania Life Insurance Building and the Ritz-Carlton,
Vanderbilt Hotels; in Cleveland, Ohio, the Rockefeller Buildin
Mass., the Jordan-Marsh Building; in Duluth, Minn., the So
Worcester, Mass., the Worcester County Court-House; and in
Cal., the First National Bank Building. In some of the buildi
in the preceding articles, HOLLOW METAL doors, trim and moldi
panied by bronze or other METAL-COVERED-WOOD window-frames

Hollow Metal Door-Frames, Trim and Moldings. Afte
METAL door reached an advanced stage of construction the
turned their attention to the problems involved in making me
moldings. It was found that moldings made by the ordinar
PROCESS were too rough and heavy and required too much la
and finish their surfaces; and that those pressed from light-gau
common methods were not clear-cut and definite in their out
limited in length and in variety of shapes. Accordingly, what is
COLD-DRAWN METHOD of making frames, trim and moldings, was
perfected, and moldings made by this process are now used for
interior work. The cold metal is drawn through special dies
required shape and the bright finish is retained. The corners a
out sharp and true and the pieces possess much greater strengt
than those HOT-ROLLED and several times thicker. There are
thousand shapes. Moldings can now be made in lengths up to
ft, but extra-freight rates and other draw-backs make it inadvis
in lengths of over 20 ft. Besides the COLD-ROLLED special hi
brass, bronze and copper are used in their manufacture. The
include angles, channels and Z bars; moldings for bases, com
duits, door-jambs, sash-bars, panels and glass; picture-fram
window-casings and trims of all kinds; wainscoting and c
numerous miscellaneous sorts. WROUGHT-IRON welded one-pie
are made for use in fire-proof partitions. These frames* a
scientifically of specially rolled WROUGHT IRON in several di
The mitered corners are welded together making the frame
They are made for any thickness or type of door or partiti
bracing and can be fitted with invisible hinges if required.

Hollow Metal Window-Frames and Sashes.† HOLLOW
frames and sashes, as well as those which are made of METAL-
and of cast iron, wrought iron, drawn bronze, cast bronze, etc., a
wire-glass, prism glass, electroplated glass, etc., are used in those
ings in which the exposure to fire is not great enough to require th

regulations have been very conveniently condensed by Mr. J. K.

ed Trim. A process recently introduced consists in electrically ayer of copper on the outer surface of wooden moldings or doors. eposit preserves the markings of the grain of the wood and makes able door. A good sample of this work has been installed in the ceering Building, New the New York Central any of the same city. work of this kind has the Hecla Iron Works City by electroplating of material known as

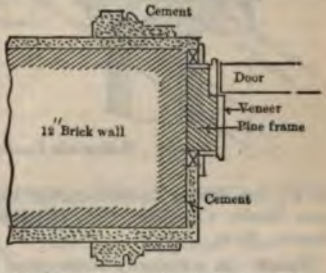


Fig. 89. Door-jamb with Cement Trim

rim. Keene's cement d for many years for moldings, door and win- , and in many European tically all of the interior s material. Any mold- s in it with good sharp is sufficiently hard to stand ordinary usage. Fig. 89 shows a with a trim of Keene's cement. This detail can be further im- ering the wooden frame and door with thin metal. The metal n be painted as desired.

ollow Tiles for Inside Finish. These are also being substituted ry wooden finish. The Amelia Apartments, erected by H. B. on, Ohio, in 1901,† is built almost entirely of hollow tile. "The ture-moldings, and the architraves around the doors were made ormed tiles, as shown in Fig. 90. These tiles were afterward rmonize with the scheme of color-decoration. All of the floors e building are covered with a cement composition composed of

even roll-top desks and highly ornamental cabinets, may now be
metal; and many libraries, banks, and court-houses have been
furnished entirely with incombustible cabinet-work. Catalogues
tained from the leading companies engaged in the manufacture of
NITURE, such for example as the Art Metal Construction Company,

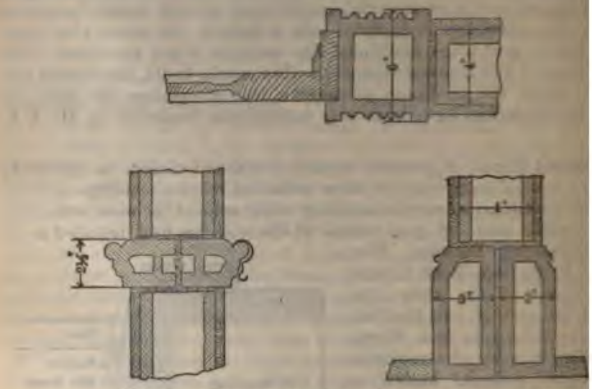


Fig. 90. Hollow-tile Door-trim, Picture-molding and Base

N. Y., the Berger Manufacturing Company, Canton, Ohio, the
Iron Works, Cleveland, Ohio and the Library Bureau, New York
Boston, Mass.

Stairs. In a majority of fire-proof buildings the architects have
themselves with putting in INCOMBUSTIBLE STAIRS of iron, with
or marble treads. As
in the first pages of this
unprotected iron cannot
sidered fire-proof, but it
to protect the ironwork
way, as it is usually
the same time preserve
mental effect. If
construction is to be
iron is much to be
steel, as the cast metal
its shape under
better than thin facings
works of steel. Slate
treads and platforms, unless supported underneath, should never be
case-construction. When subjected to heat, marble and slate crack
away, leaving the stairs impassable. A fire-department captain in
City lost his life through the collapse of a marble platform. If these
to be used, therefore, there should be a subtread of iron or concrete
A really FIRE-PROOF STAIRCASE should be constructed with as little
as possible, and what ironwork there is, incased in FIRE-RESISTING
It is possible and practicable to build stairs of clay tiles, bricks,

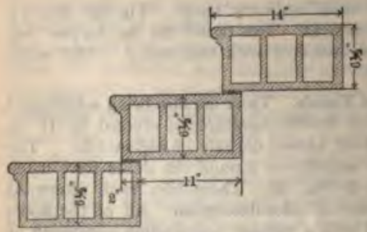


Fig. 91. Hollow-tile Steps for Staircase

ny has
l stair-
ing to
of con-
ing flat
bedded
No iron-
ver is
nstruc-
it is
e-proof.
s a par-
f a tile
as was
Amelia
uilding,
The
f hard-

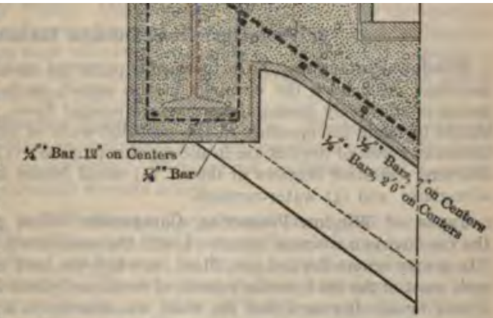
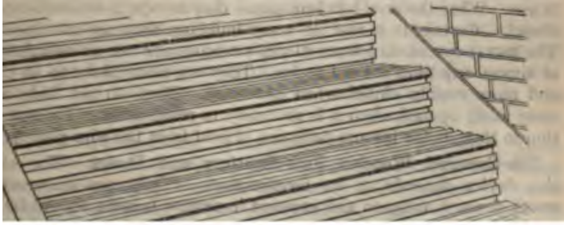


Fig. 92. Reinforced-concrete Stairs, Government Printing Office, Washington, D. C.

rial, glazed, and 4 ft long. They were supported upon the s and were used by the mechanics for carrying up material erection of the building. Reinforced concrete, with slate or s, is a good material for the construction of stairs and permits



bedded in the mortar if desired. (See, also, pages 951 and 98

9. Protection from Outside Hazard

Window-Protection. To be thoroughly protected against th buildings must have the openings in the outside walls pro means of effectively closing those openings against flame. Th should be made for openings in the partition-walls of large GENERAL TYPES of devices are in use for this purpose: (1) tin shutters; (2) steel shutters or doors; (3) metal frames and wire-glass; and (4) water-curtains.

Types of Window-Protection Compared. When prog the TIN-COVERED WOODEN SHUTTER is still the most effective wi "In a very severe fire in Lynn, Mass., in which the heat was i melt most of the tin from the outside of the tinned plates cover it was found afterward that the wood was charred to a dept ⅜ in. The shutters were warped slightly, but afforded suf against the heat to allow men to remain behind them to put occasionally crept through. This would not have been poss shutters under similar conditions."* STEEL SHUTTERS, und heat, warp very readily and transmit considerable heat. Th cheapest type of window-protection. "There is one objecti shutters on window-openings and that is, that they depend o agency to be effective. They must necessarily be open whil in use. When the need for them comes they are apt to be overlo closed. Certain it is that on many buildings they are not cl The METAL-FRAME-AND-WIRE-GLASS WINDOWS are not as unsig of almost any kind are apt to be. They are more likely to b and more readily closed when necessary. They do not hid more easily opened when it is necessary to reach a fire. The o tion to them is the intense radiation of heat from the wire-glass

Tin-Covered Wooden Fire-Shutters and Doors. The this device depends on its construction. "Only well-seaso ı is about ¾ in. Under intense heat the wood is certain to char, but are long enough to hold the tin up against the wood, and the tin put on so as to keep the air out to prevent burning, the shutter under severe strains."[*] The hinges, fastenings, or hangers must be the door, not nailed or screwed, as nails or screws would pull out re. If hung on hinges, the hinge-hook should be built into the wall. was designed for use in mills, but it has worked so satisfactorily generally adopted wherever a fire-proof door is wanted and its ap- s not objectionable. Fire-proof shutters, also, are made in this way. nal Board of Fire Underwriters issues complete specifications[†] for of door and shutter, and these specifications should be closely fol- satisfactory results. Doors of this type, provided for the openings partition-walls, are often, and wherever possible should be, hung on acks so that they will close automatically. Where it is desirable to open most of the time, an automatic release operated by a fusible vided. (See, also, page 778.)

Covered Wooden Doors as Fire-Doors. Wooden doors covered LAMEIN or other process (page 899) are sometimes used as fire-doors earance is a consideration. They are not considered equal, however, NDARD TIN-COVERED WOODEN DOORS.

ire-Doors and Shutters. For a satisfactory STEEL FIRE-DOOR eet of steel should be used, and it should be reinforced on the back me of angle-irons, not less than 1½ by 1½ by ¼ in, and increasing in the door or shutter. These doors or shutters may operate in one of s: (1) swing on hinges, (2) slide on tracks, or (3) roll vertically. GING DOORS or shutters are the most reliable as there are no com- arts to get out of order. They should be hung on eyes built into the walls. SLIDING DOORS or SHUTTERS must have the rails on which ate protected by metal shields to prevent obstruction. For larger the ROLLING SHUTTERS are generally preferred. They are made in jointed sectional strips, which wind up on a roller placed in a pocket opening, the ends moving in metal grooves to hold them in place. rally operate VERTICALLY, although some are made to operate HORI- , the rollers being set vertically in pockets at the sides of the openings. er are more apt to get out of order. The VERTICALLY operated doors s are balanced by springs or weights to make them move easily up Where they are intended to be closed in case of necessity only, they y weighed and held open by means of fusible links, so that in case y will close automatically.

Metal for Fire-Resisting Window-Frames and Sashes.[‡] These ade weather-tight and perfectly practicable in all respects, and should wherever fire-resisting windows are desired. The sashes are made for holding wire-glass. These SHEET-METAL WINDOWS are made in riety of forms to meet all purposes and the sashes may be stationary, ither horizontally or vertically, hinged, or double-hung with weights, ary windows. For factories, warehouses, stairways and elevator- tationary lower and a pivoted upper sash are commonly used, as this apest type of window. The double-hung windows are now made s smoothly as wooden sashes in ordinary box frames. For offices,

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[*] Insurance Engineering, Dec., 1902.

[†] To be had for the asking.

[‡] See, also, Hollow Metal Window-Frames and Sashes, *page 902.*

hotels, etc., a window having two sashes, glazed with wire-gl
and locking automatically in case of fire, and a third inner s
clear glass, has all of the advantages of an ordinary window wit
advantages of fire-protection and better diffusion of light. M
also, can be used with these windows. All movable sashes, gl
glass, should be provided with a device by which the sashes
lock automatically in case of fire. When the contents of a
flammable and the exposure severe, two thicknesses of wire-
used with a ventilated air-space of at least 1 in between the li

10. Extinguishing Devices and Precautionary M

Water-Curtains. "The vulnerable portion of buildings
front, where great window-openings are desired for purposes of
it is considered objectionable on account of appearance to h
even wire-glass windows. These large window-openings affor
tunities for the spread of fire across streets. The danger of
increased where the fronts, as is very common, are made of un
work. A notable example, illustrating such danger, was the
Manhattan Savings Institution, New York City, which was se
and almost destroyed by a fire in a six-story non-fire-proof bui
street. Such conditions might be overcome to some extent
introduction of some system such as the WATER-CURTAINS that
the Chicago Public Library. This is practically a SPRINKLER-S
the edge of the cornice of the building, and so arranged as t
sheet of water in front of the building. Such a sheet will, how
far before it is turned into spray and thus becomes practically us
arrangement placed at each window-opening might be more u
is doubtful whether it would be of much value in any severe
The rules of the National Board of Fire Underwriters for OP
or water-curtains determine the size of piping, feed-mains
arrangement of the system.

Precautionary Measures in General.† No matter how tho
ing is fireproofed, if it is filled with combustible goods, as a wa
factory, there is always the possibility of a fire, which, if unch
started, must necessarily entail a great loss and more or less
building. If a fire is discovered and checked in its incipient
avoided. There are now many valuable devices for DETECTING
fires, which should be installed in every warehouse, and whic
placed with advantage in buildings used for other purposes. The
of these are: automatic alarms, automatic sprinklers and st
reels, etc.

Automatic Alarms. By means of very sensitive therm which forms one side of the circuit has a thick coating or wall of al; over this is an insulating coating. A number of fine wires are this coating to form a second conductor. The whole is then covered le insulation. When flame or a dangerous degree of heat comes into h this wire it establishes electrical connection between the two con- gives a signal on the premises equipped.

solidated Fire Alarm Company, of New York City, controls a device AUTOMATIC FIRE-ALARMS, which consists essentially of an air-tight of about ⅛ in internal diameter, inserted every 10 ft along a main . At the end of each tube is attached a metal cylinder with a dia- ld by a soldered coiled spring. Under the unusual heat of a fire, plug melts, releases the spring which actuates the diaphragm and npulse of air through the tube. This air-hammer operates a regular n-device for sending an electric alarm to a central station where f the alarm can be determined by the nature of the signal. If caused he signal is relayed to the fire headquarters. At the same time, an r, usually located in the first story of the building, indicates the which the original impulse was given. The soldered-spring thermo- cated in accordance with the requirements of the National Board of writers.*

erican District Telegraph Company and the International Electric Company of New York City control systems which utilize the prin- e EXPANSION OF AIR UNDER HEAT. A small copper tube which is nd air-tight is installed in one continuous length along the ceilings At the end of its tour of a room this tube terminates in a box con- sensitive diaphragm. When the temperature of a room reaches a e of from 150° to 155° F., at an undue speed, 4° or 5° per minute, the tube expands and sets up a pressure on the diaphragm, which lectric circuit. False-alarm vents are provided in the diaphragm so ht rise in temperature brought about by natural causes is taken care air allowed to escape without sending in an alarm.

ctric Fire Alarm System, as installed and controlled by the Con- Fire Alarm Company of New York City, consists of two continuous t electric circuits with soldered coiled-spring thermostats distributed nce with the Underwriters' requirements. Should contact be broken t, in either one of the two circuits, the transmitter is tripped and false alarm to the central station, thus serving as a TROUBLE-SIGNAL rating company and indicating that the system is out of order. In fire, both fuses go, breaking both circuits, and thus transmitting the a REAL FIRE through the central station to the fire department. At ime, an annunciator in the station picks out the particular story of g from which the signal was sent.

tic Sprinklers. "An AUTOMATIC SPRINKLER is a device for distribut- by means of a valve which is arranged to open under the action of om a fire which it is intended to extinguish. The distribution of ch results from properly located sprinklers occurs in the form of a or drops, and is sufficient to drench almost any inflammable stock point of ignition. The distribution is also economical, as the water nly applied than from a nozzle attached to a fire-hose, and the source above the fire. Whenever combustible merchandise constitutes ts of a building, AUTOMATIC SPRINKLERS are of great value, and in f a height so great as to make the upper stories difficult of access,

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g Systems Recommended by the *National Fire Protection Association*, 1911.

from 155° to 165° F., and the HARD-HEAD SPRINKLERS which
to 300° F. for engine-rooms and boiler-rooms. Sprinkler-hea
of even higher melting-points are sometimes used. The fo
been approved by the National Board of Fire Underwriters
ratings: the Evans Variable Pressure Model, International S
of New York City; the Grinnell Type, General Fire Exting
Providence, R. I.; and the Manufacturer's Type, Automat
pany of America, New York City. "No appliances meeti
ments desirable for this service have as yet been shown." †

Sprinkler Supervisory Devices. These devices consist
"TRANSMITTING SIGNALS when gate-valves are closed or op
tanks falls below or is restored to a predetermined level;
air-tanks falls below or is restored to a predetermined am
in tanks falls below or rises above predetermined temperatur
water-flow signals and to withhold signals from water-surge
sures." They are used in connection with CENTRAL-STA
SYSTEMS for supervising the operation and maintenance
ments. The A. D. T. devices, manufactured by the Automa
Company of New York City and Chicago, Ill., are approve
Board of Fire Underwriters.†

Stand-Pipes and Hose-Reels. In office-buildings, hote
houses, where sprinkler-systems are hardly suitable, STAND
REELS in each story and on the roof, ready for instant use,
means of quickly controlling a fire. The stand-pipe should
in diameter, according to the size and height of the building,
nected with the water-supply of the building and provided
nections at the street-level for the fire department. Chec
provided, so that when the fire-department engines are at
will be added to the force due to the head of water from the
fire-pumps or to the force of the city water-system.

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* J. K. Freitag.

† List of Fire Appliances, National Board of

CHAPTER XXIV

NFORCED-CONCRETE CONSTRUCTION *

By
RUDOLPH P. MILLER
ENT OF BUILDINGS, BOROUGH OF MANHATTAN, NEW YORK CITY

1. Introductory Notes

. The term REINFORCED CONCRETE is defined in the regulations of k City Bureau of Buildings as "an approved concrete mixture re- eel of any shape." The Philadelphia law has the same definition, tion, "so that the steel or iron will take up all the tensional stresses he resistance to compression and shear."

Notes. The great value of concrete as a structural material when compression only has been recognized for centuries. The use of ncrete, however, as a practicable and commercial form of construc- aratively recent. It is true that as far back as 1869, Francois ris took out letters patent on a combination of iron and concrete before this, in 1867, the principle of reinforcing concrete with iron lied by P. A. J. Monier, a gardener of Paris, to the making of large still, the general application to building-construction did not occur middle of the last decade of the nineteenth century. In its develop- irst applied to bridge-construction. The discussion of the subject er is confined to its use in the construction of buildings. The ple of a building of reinforced concrete in this country, and probably is that erected in 1875 by W. E. Ward, near Port Chester, N. Y., t only all the external and internal walls, cornices and towers were f concrete, but all of the beams and roofs were exclusively made of orced by light iron beams and rods." †

ion of Reinforced-Concrete Work. In general outline, a build- in reinforced concrete consists in the usual preparations of the site n or otherwise, the provision of suitable foundations for walls, her supports, the erection of a series of wooden molds or forms, the e necessary steel reinforcement, the pouring of the concrete and of the forms after the concrete has set sufficiently to sustain itself that may come on it during construction. From the beginning n of the forms the successive steps are progressive, that is, the steel and pouring of the concrete are going on in the lower sec- ies while the forms are being erected for the upper sections or hat in a large operation the carpenters, the steel-setters and the y all be working at the same time, one set slightly in advance of hout interference one with the others. These several steps in the considered in greater detail in Chapter-Subdivision 7, page 962, einforced-Concrete Construction.

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ete in general and Mass-Concrete, see Chapter III, pages 240 to 251; for ncrete without Reinforcement, Chapter V, pages 283 to 287; and for crete Factory-Construction, Chapter XXV. See, also, Chapter XXIII, 44.

er and more extended history the reader is referred to the larger treatises and to Edwin *Thacher's article in Engineering News, March 26, 1903.*

2. Materials Used in Reinforced-Concrete Con

The Materials used in reinforced concrete are CONCRETE
concrete forms the mass of the construction. Its proper us
pression. While it has some tensile strength the amount i
variable that it should always be neglected. STEEL is used
material as it furnishes the greatest amount of strength at
WROUGHT IRON could be used, but it is practically unobtain
conditions, and, as already intimated, its use is not economic

Concrete. The CONCRETE consists of a mixture of cemen
gate, in definite proportions, with the necessary water to caus
cement.

Cement. PORTLAND CEMENT should always be used in r
and it should always be tested before being used. Even in
portant to know that the cement is strong and sound. In pur
the certificate of some reliable testing-laboratory should b
conditions of acceptance. Under all circumstances, it is al
the testing done at some well-established and properly equip
laboratory. The results of tests in temporary laboratories
and may lead to unnecessary controversies with the man
acceptable, a cement should meet the following requireme
the standard specifications of the American Society for Testi

SPECIFIC GRAVITY. The specific gravity of the cement,
100° C., shall be not less than 3.10.

FINENESS. It shall leave by weight a residue of not more
100, and not more than 25% on a No. 200 sieve.†

TIME OF SETTING. It shall develop initial set in not less th
must develop hard set in not less than 1 hour, nor more tha

TENSILE STRENGTH. The minimum requirements for
briquettes 1 in square section shall be as follows, and shall sh
in strength within the periods specified:

NEAT CEMENT

24 hours in moist air. . . . .
7 days (1 day in moist air, 6 days in water). . . . .
28 days (1 day in moist air, 27 days in water). . . . .

ONE PART CEMENT, THREE PARTS STANDARD OTT

7 days (1 day in moist air, 6 days in water). . . . .
28 days (1 day in moist air, 27 days in water). . . . .

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* For the complete standard specification, recommended Augu

RIC ACID AND MAGNESIA. The cement shall not contain more than
nhydrous sulphuric acid (SO_3) nor more than 4% of magnesia (MgO).
OF CONSTANCY OF VOLUME OF SOUNDNESS is of particular importance
ced-concrete work. When used in large masses an occasional batch
made with unsound cement may not seriously affect the final result,
forced-concrete building operations, where the different members of
ures are comparatively small, the safety of the entire building may
ized by the use of a small amount of unsound cement in some impor-
such as a column.

ate.† By the term AGGREGATE is understood the materials, including
nixed with the cement to make the concrete. In practically all cases,
a necessary element.

"The SAND should be clean. One may obtain some idea of its cleanli-
cing it in the palm of one hand and rubbing it with the fingers of the
the sand is dirty, it will discolor the palm. If the use of dirty sand
able, its effect upon the strength of the mortar should be investigated.
should be given to sand containing a mixture of coarse and fine
xtremely fine sand can be used alone, but it makes a weaker mortar
r coarse sand alone or a mixture of coarse and fine sand."‡ Mortars
of one part Portland cement and three parts fine aggregate or sand,
, should show a tensile strength of at least 70% of the strength of
ar of the same consistency and of the same cement mixed with
Ottawa sand. The New York Regulations specify that fine aggregate
ist of sand, crushed stone or gravel screenings, passing when dry, a
ing ¼-in-diameter holes, and passing not more than 6% through a
ng 100 meshes per linear inch. The Chicago regulations specify that
han 45% shall be retained on a screen of 400 meshes to the square
e, also, page 241.)

Aggregate. For the COARSER MATERIAL OF THE AGGREGATE many
are used and many others have been suggested. Its selection is gen-
endent upon local conditions. If possible, gravel or crushed stone
used. Whatever is used should be a clean, hard substance that will
the concrete the necessary strength; that is, the crushing strength
terial should be equal to or greater than that of the mortar used, at
age of 28 days. In any case, where no reliable information is to be
strength of a concrete made from a given aggregate, careful investi-
uld be made before such material is used. (See, also, page 241.)

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ceding foot-note relating to Fineness, page 912.
so, Chapter III, pages 240 to 251. The data there on Aggregates, Propor-
terials, etc., relate more particularly to mass-concrete, while the data of
XIV is intended to cover, more in detail, reinforced concrete.
on Concrete, Plain and Reinforced, Taylor and *Thompson*.

if the soft varieties are excepted, make excellent concrete a
concerned. They would, however, seem to affect the fir
the concrete. (See Tables on page 957.) The harder
SANDSTONES, also, may be used successfully, but great car
in their selection. CONGLOMERATE, which is in reality a har
should give very satisfactory results. On account of their lo
SLATE or SHALE should not be used in concrete. Besides
mentioned, broken BRICK, TERRA-COTTA, FURNACE-CLINKER
have been suggested. In the selection of broken brick or te
be taken to get hard-burned material. The crushing streng
when well selected, is a little more than that of acceptabl
old. But ordinarily, commercial brick or terra-cotta will n
ments for a good aggregate, and these materials should be
resort and then only after careful investigation. (See, also,

Cinders. FURNACE-CLINKERS should be clean and enti
bustible matter. CINDERS are often used where fireproo
consideration, and no doubt good constructions may be obt
care, by the use of clinker or cinder concrete, especially if th
screened and graded as suggested for gravel. But in gener
crete is not uniform in quality and is unreliable in strength.
considered in this chapter. In Chapter XXIII, Fireproofi
use is discussed on page 818. (See, also, page 242.)

Size of Aggregate. The SIZE OF THE AGGREGATE may
in in largest diametrical dimension, depending on the par
which it is to be used. Where the mass of concrete is com
aggregate may run as high as 2½ in in size. This may so
in foundations and in large piers and thick walls. In colu
and slabs, very unsatisfactory results would be obtained if s
used. For such work no stone or other aggregate should
would pass a 1-in screen. In important girders and colum
the reinforcing-bars are closely spaced, the size should be m
that a concrete of viscous consistency is produced "which
tween and easily surround the reinforcement and fill all por

the strongest. The old method of determining the proportions of
measuring the voids in the coarser particles by means of water
a box containing 1 cu ft of the material and then providing that
finer material, assuming the cement the same as sand, is not to be
ed. It does not give accurate or satisfactory results. A better
to take the materials to be used and make trial-mixtures by varying
ions, always using, however, the same amount of cement and water.
-mixtures are placed successively in a measuring vessel of fixed size
d, and the height to which the vessel is filled for each mixture is
e proportions that give the lowest height, or result in the smallest
ll give the most satisfactory concrete. (See, also, page 242 and fol-
es.)

and most scientific method, however, is that known as the MECHAN-
SIS, devised by W. B. Fuller. In this method the available materials,
he cement, are separated into the various sizes by means of a series
urves are plotted which indicate the percentages of the whole mass,
the several sieves; and from a study of these curves the proportions
rent aggregates are determined. For a detailed description of this
reader is referred to the chapter on Proportioning Concrete in the
n of the Treatise on Concrete, Plain and Reinforced, by Taylor and
As an example of the saving possible, the following case, given in
st referred to, will be of interest:
linary mixture for water-tight concrete is about 1 : 2½ : 4½, which re-
barrels of cement per cubic yard of concrete. By carefully grading
ls by methods of mechanical analysis the writer has obtained water-
with a mixture of about 1 : 3 : 7, thus using only 1.01 barrels of cement
rd of concrete. This saving of 0.56 barrel is equivalent, with Port-
t at $1.60 per barrel, to $0.89 per cu yd of concrete. The added cost
r proportioning and mixing the concrete, because of the use of five
ggregate instead of two, was about $0.15 per cu yd, thus effecting a
of $0.74 per cu yd. On a piece of work involving, say, 20 000 cu yd
, such a saving would amount to $14 800, an amount well worth
e study and effort on the part of those in responsible charge."
rdinances or regulations governing reinforced concrete of various
roportions to be used are generally prescribed. In New York, "the
r reinforced-concrete structures shall consist of a wet mixture of one
ent to not more than six parts of aggregate, fine and coarse, either in
ions of one part of cement, two parts of sand and four parts of stone
r in such proportion that the resistance of the concrete to crushing
less than 2 400 lb per sq in after hardening for 28 days." In Chicago,
les of concrete are specified with the ultimate compressive resistance,

gress Report, Proc. Am. Soc. C. E., Feb., *1913, pages 137 and 138.*

to be developed, from a mixture of 1 : 1 : 2 and an ultimate strength
per sq in, to a 1 : 3 : 7 mixture with a strength of 1 500 lb per sq in.
and San Francisco the proportion is given as one of cement to six o
In Buffalo a 1 : 2 : 5 mixture is required.

Compressive Strength of Reinforced Concrete. For reinforc
work no mixture should be used that does not develop a COMPRESSIV
of at least 2 000 lb per sq in at the age of 28 days. The crushing
various concretes is shown in the following table:

Table I. Compressive Strength of Portland-Cement Concr
Different Proportions

Proportions			Age, months	Compressive strength per sq in	Authority
Cement	Sand	Stone			
1	1	0	4	4 370	James E. Howard, Tes Arsenal
1	2	0	4	2 506	
1	3	0	4	1 812	
1	4	0	4	830	
1	5	0	4	532	
1	6	0	4	169	
1	7	0	4	118	
1	2	4	4	2 178	
1	3	6	4	1 815	
1	4	8	4	1 135	
1	5	10	4	707	
1	6	12	4	738	
1	2	2	4	1 768	
1	2	3	4	1 911	
1	2	4	4	2 147	
1	2	5	4	2 452	
1	2	6	4	2 124	
1	2	7	4	1 650	
1	2	8	4	1 295	
1	2	4	1	2 399	G. A. Kimball, Tests of
1	2½	5	1	3 255	Taylor and Thompson, town Arsenal
1	3	5	1	2 042	Watertown Arsenal, Te U. S. A.

Working Stresses for Reinforced Concrete. Some for
strength of reinforced-concrete construction provide for the use of
STRENGTH of the concrete and the application of a FACTOR OF
practice is not to be recommended as it necessitates either the t
crete or the assumption of an ultimate strength. While it is un
sirable that the concrete should be tested, this is generally impra
the building is being designed. It should be done during constr
done on the best work, to make sure that the concrete is up t
ments. Various factors of safety from two and one-half to ten h
posed. Different factors of safety are used for different members
or for different conditions. This is another reason why it would b
WORKING STRESSES than ULTIMATE STRESSES. The following WORK
are recommended for reinforced concrete that will develop a CRUSH
of 2 000 lb per sq in in 28 days:

gh Steel. Two grades of steel are used for the reinforcement,
d HIGH-CARBON STEEL. MILD or MEDIUM STEEL is used for all
es and is the ordinary MERCHANT-STEEL. It has an ultimate
of from 60 000 to 70 000 lb per sq in, and its elastic limit is about
timate strength. HIGH-CARBON STEEL has a greater percentage
is therefore more brittle. Its ultimate strength is about 105 000
imit about 55 000 lb per sq in. The use of HIGH-CARBON STEEL
greater stresses in the reinforcement, and consequently a less
and a greater economy in construction. On account of its greater
ever, it is liable to sudden failures under stress. It is also often
cked or broken when sent to the work, and unless it is very care-
there is great liability of defective material getting into the
rthermore, much of the so-called HIGH-CARBON STEEL has been
ce, after testing, to fall far short of the specifications. Its use is
avoided, unless special care is taken to secure an absolutely re-
id to have it inspected and tested. For large, important work
esirable. Ordinarily, however, mild steel should be used, as com-
manufactured and sold under such standard conditions that it is
e modulus of elasticity of high-carbon steel is practically the same
dium steel, the deformation under any given loading is the same
special advantage in the use of one over the other. Steel filling
ts of the specifications adopted by the American Railway Engi-
aintenance of Way Association is recommended by the Commit-
erican Society of Civil Engineers. This specification calls for a
TE TENSILE STRENGTH of 60 000 lb per sq in and should not vary
o lb per sq in from the desired ultimate strength. For slab and
nforcement where wire or small rods are suitable, steel manu-
Bessemer billets may be used with a TENSILE STRENGTH of 105 000,
INT of not less than 52 500 lb per sq in.

resses for Steel. The generally accepted WORKING STRESS for
16 000 lb per sq in in tension. Tests have shown that in cases
re of reinforced-concrete beams is due to the failure of the rein-
stress in the metal had not more than reached the YIELD-POINT.

Table II. Working Stresses for Reinforced-Concrete Construction

Authority	Extreme fiber-stress, concrete in compression, lb per sq in	Direct compression in concrete, lb per sq in	Shearing-stress in concrete, lb per sq in	Shearing-stress in concrete when all diagonal tension is resisted by steel, lb per sq in	Adhesion of steel to concrete, lb per sq in	Tensile stress in steel, lb per sq in	Tensile stress in steel to resist diagonal tension, lb per sq in	Ratio of modulus of elasticity of steel to that of concrete
New York............	650	500	40	150	80	16 000	16 000	{ 15 (girders) { 12 (columns)
Chicago.............	35% ult.*	⅓ ult.*	1/50 ult.*	1/15 ult.*	70	{ ⅓ E. L.† { ≯18 000	{ shearing-stress 12 000 }	15
Philadelphia.........	600	500	75	....	50	16 000	....	12
Boston..............	500	{ 416 max. { 347 min.	60	....	60	16 000	....	{ 15 (girders) { 10 (columns)
Cleveland...........	700	500	40	125	50 to 100	16 000	10 000	15
Baltimore...........	500	400	50	....	....	15 000	10 000	15
Detroit..............	650	450	40	....	40	{ 45% E. L.† { ≯16 000	....	15
Buffalo..............	500	350	50	....	50	16 000	10 000	12
	[illegible]	[illegible]	75	....	60	20 000	10 000	15

Table III. Specifications for Reinforcing Steel

ll be rolled from new billets to meet the following Manufacturers'
fications:

onsidered	All steel except alternate for deformed bars	Alternative specifications for deformed bars only	
		Structural grade	Hard grade
....	Open hearth	Open hearth or Bessemer	Open hearth or Bessemer
naximum ..	0.06	0.10	0.10
ile'strength, square inch	60 000 to 70 000	55 000 to 65 000	80 000 (minimum)
minimum square inch	60% T. S.	33 000	50 000
minimum 3 in........	$\frac{1\ 400\ 000}{\text{tensile strength}}$	$\frac{1\ 250\ 000}{\text{tensile strength}}$	$\frac{1\ 000\ 000}{\text{tensile strength}}$
-tests, with-	180°, to a diam equal to the thickness of piece tested	Bars under ¾ in, 180°, $d=1\ t$; ¾ in and over, 180°, $d=2\ t$	Bars under ¾ in, 180°, $d=4\ t$; ¾ in and over, 90°, $d=4\ t$

embers. Reinforcement is used in a variety of shapes and com-
ly all of them patented and some of them forming the basis for
MS. Where the reinforcement is employed to take up tension,
girder, the BOND between the concrete and the steel is relied upon
TENSIONAL STRESSES in the steel. The plain bars depend entirely
SION of the steel and the concrete for the action of the two mate-
ation, or the full tensile strength of the rod is developed by anchor-
to the concrete at the ends, in which case the beam becomes more
trussed beam with the rod as the tension-member. In cross-
bars are usually round or square, though sometimes flat bars
other shapes are used. In regard to the use of square bars and
pes, it is contended that the edges start initial cracks in the con-
nks in setting. Twisted flat bars, when placed too near the sur-
crete, cause a spalling or breaking out of the concrete from between
ns, when the steel is under stress.

Bars. With the DEFORMED BARS the adhesion of the concrete to
plemented by a MECHANICAL BOND due to the shape of the bar.
deformed
n and are
ely used.

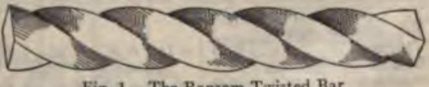


Fig. 1. The Ransom Twisted Bar

ome Bar. Twisted
are made
, twisted cold. The work on the bars in the twisting process in-
tic limit and the tensile strength; but the amount of the increase
variations in the grade of rolled steel may result, after twisting,
variations. The users of this bar generally assume a working
lb per sq in. The patent on this bar has expired and it may now
one. Strictly speaking, this is not a deformed bar. These bars

can be obtained in all sizes, varying by ⅛ in from ¼ to 1⅛ in. Larg also, can be obtained on special order.

The Spiral Steel Bar. The Buffalo Steel Company of Tonawand makes a round b longitudinal spiral tions, thus elimin sharp corners (It is made in size ⅜ to 1¼-in diam

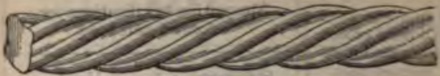


Fig. 2. The Spiral Steel Bar

the cross-sectional areas are equal to the areas of equivalent squar bars are rolled from old railroad rails and twisted hot. They are made carbon steel with an elastic limit of from 65 000 to 80 000 lb per sq in.

Corrugated Bars. Corrugated bars (Fig. 3), both square and round in cross-section, are made by the Corrugated Bar Company of New York City and Buffalo, N. Y., of both medium and high-elastic-limit steel with a yield-point of about 50 000 lb per sq in. Corr-Bars are furnished either straight and cut to length, or bent ready for the forms. The sizes are as follows:

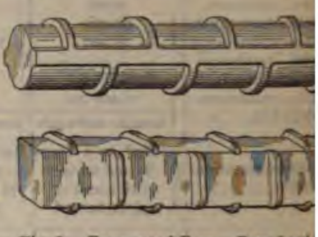


Fig. 3. Corrugated Bars. Round and

CORRUGATED ROUNDS

Size in inches	⅜	½	7/16	⅝	¾	⅞	1
Net area in square inches	0.11	0.19	0.25	0.30	0.44	0.60	0.78
Weight per foot in pounds	0.38	0.66	0.86	1.05	1.52	2.06	2.69

CORRUGATED SQUARES

Size in inches	¼	⅜	½	⅝	¾	⅞	1
Net area in square inches	0.06	0.14	0.25	0.39	0.56	0.76	1.00
Weight per foot in pounds	0.22	0.49	0.86	1.35	1.94	2.64	3.43

The Havermeyer Bar. The Havermeyer Bar (Fig. 4), control Concrete Steel Company, Youngstown, Ohio, consists of square and rolled with a series of gradual projections on all sides, the deformat so designed that there is a constant cross-sectional area. They are f the following sizes and weights:

hes	Square bars		Round bars	
	Area in square inches	Weight per foot in pounds	Area in square inches	Weight per foot in pounds
	0.0625	0.212	0.0491	0.167
	0.1406	0.478	0.1104	0.375
	0.2500	0.850	0.1963	0.667
	0.3906	1.328	0.3068	1.043
	0.5625	1.913	0.4418	1.502
	0.7656	2.603	0.6013	2.044
	1.0000	3.400	0.7854	2.670
	1.2656	4.303	0.9940	3.379
	1.5625	5.312	1.2272	4.173
	1.8906	6.428	....	....
	2.2500	7.650	....	....

of 5% under and 2½% over the above weights is required for rolling.

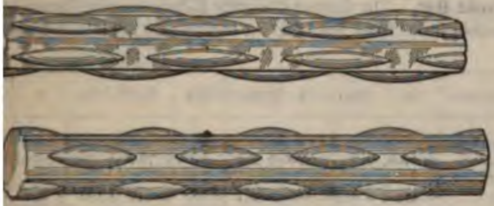


Fig. 4. The Havermeyer Bar, Square and Round

mond Bar. The Diamond Bar (Fig. 5), put on the market by the eel Engineering Company, New York City, is claimed as the only bar of uniform ncre is y no tal due eforma- re is in This ically a round bar, and as sudden transitions from one section to avoided, all tendency to cause initial cracks in the concrete is overcome. The weights and areas of Diamond bars are equal to those of plain square bars of like denominations. Bars from ¼ to 1¼ in in diameter may be obtained from the Concrete Steel Engineering Company, of New York City.

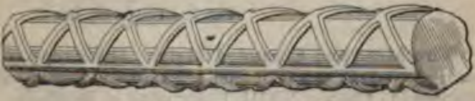


Fig. 5. The Diamond Bar

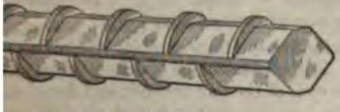


Fig. 6. The Rib-bar

Bar. The Rib-Bar (Fig. 6) manufactured by the Trussed Concrete ny of Detroit, Mich., is a rolled section with a series of cross-ribs.

It is of rectangular section and furnished in sizes of from ¼ to 1¼ in., the
of the cross-sections being equivalent to squares of equal denominatio
the weights are slightly greater, and are as follows:

Size in inches	Area in square inches	Weight per linear foot in pounds
¼	0.0625	0.213
⅜	0.1406	0.48
½	0.2500	0.86
⅝	0.3906	1.35
¾	0.5625	1.95
⅞	0.7656	2.65
1	1.0000	3.46
1⅛	1.2656	4.38
1¼	1.5625	5.41

The Ovoid Bar. The Gabriel Concrete Reinforcement Company of De
Mich., furnishes the Ovoid Bar (Fig. 7), in sizes and areas as follows:

Size in inches	⅜	½	⅝	¾	13/16	⅞	15/16	1	1⅛
Area in square inches.......	0.1406	0.25	0.3906	0.5625	0.6602	0.7656	0.8789	1.00	1.265
Weight in pounds......	0.494	0.873	1.356	1.947	2.282	2.643	3.031	3.446	4.354

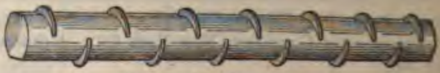


Fig. 7. The Ovoid Bar

Wire Mesh and Expanded Metal. Other types of tension-reinforc
such as WIRE-MESH FABRIC and EXPANDED METAL in various forms hav
discussed in Chapter XXIII, Fireproofing of Buildings. Wire fabric ha
into very general use as a slab-reinforcement, as it resists temperature
and the cracking of the concrete from impact or shock. It is made in
gauges with heavy longitudinal or carrying wires and lighter transver
tributing or tie-wires. Expanded metal is similar to wire mesh in pr
reinforcement in both directions, rigidly spaced and attached or faste
gether. This additional advantage is claimed for it; it provides reinfor
in all directions, thus taking care of concentrated loads.

Anchoring. Different methods have been used for ANCHORING the
bars in reinforced concrete. In the Hennebique system of construction
where plain bars are used, the ends of the rods are split and flared out.
constructions the ends of the bars are simply turned at right-angles
direction as is most suitable. In some instances nuts and washers ha
placed at the ends of reinforcing-rods. Where reinforced-concrete fl
used in connection with steel columns the rods are run through the we
or through angle-brackets and secured with nuts.

ion. The strengths of the BOND between concrete and steel for various
bars and differing conditions are shown in Table IV. After the bond
, the reinforcement still acts in conjunction with the concrete, due to
or FRICTIONAL RESISTANCE. Numerous tests have shown this fric-
istance to be about two-thirds of the initial bond-strength. The
ENGTH for ordinary round or square-section bars may be taken at 200

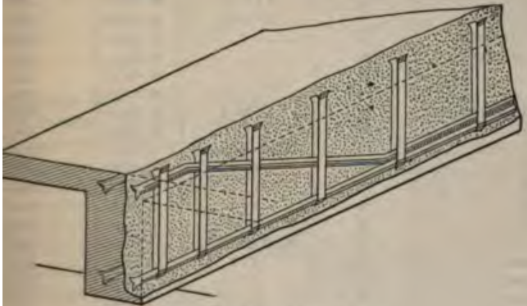


Fig. 8. The Hennebique System

per sq in, depending upon the character of the concrete and the degree
ess of the steel. MECHANICAL BOND depends upon the shape of the
he compressive and shearing strength of the concrete.

-Members. In many of the tests on full-sized concrete beams, failure
the development of diagonal breaks near the supports. The first
crack in a beam, with nothing but horizontal tension-steel at the
s apt to occur when the maximum VERTICAL SHEAR is from 100 to 200 lb
Since the vertical shear is accompanied by a HORIZONTAL SHEAR of
ensity in all parts of the beam, it was formerly thought that this diag-
re was due to these shearing-forces at the end of the beam and vertical
or bent-up rods were provided to resist the horizontal shear. More
ts have shown that the SHEARING STRENGTH of concrete is from 60 to
he compressive strength, and that these cracks are diagonal and in the
which could be expected from the THEORY OF DIAGONAL TENSION,
ributes them to a combination of the shearing-stress with the hori-
sile stress. The inclined cracks which first appear are due to a rupture
ncrete in tension. The most effective way to prevent this rupture is
e reinforcement in the direction of the stress that is inclined upwards
he supports, as nearly as possible normal to the line of the diagonal
Vertical reinforcement could be used, but it would not act until def-
or downward displacement of the concrete occurred on the side of the
y from the support. If vertical stirrups are used for this reinforce-
y must be spaced a less distance apart than the effective depth of the
d they must be looped around, though not necessarily attached to, the
l bars. When inclined reinforcement is used, it must be rigidly at-
the longitudinal members and spaced a less distance apart than the
depth of the beam. The reason for this is that the magnitude and in-
of the diagonal tension increases from the middle toward the end of the
ng inclined 45° where the horizontal *tension becomes zero.*

Table IV. Results of Tests on Adhesion Between Concrete and Ste

Kind of bar	Size tested in fraction of inch	Concrete	Age	Ultimate stre developed i per sq in of face in cont
Round	½	1 : 2 : 4	60 days	412 (a)
Square	¾	1 : 3 : 6	30 days	274 (b)
Square (rusted)	¾	....	30 days	435 (c)
Square (rusted)	¾	....	90 days	642 (c)
Square	⅞	....	90 days	431 (c)
Square	⅞	....	30 days	294 (c)
Twisted (Ransome)	⅝	1 : 2 : 4	31 days	648
Twisted	¾	....	25 days	500 (c)
Twisted	¾	Neat cement	7 mos.	1 290 (e)
Twisted	¾	1 : 1	7 mos.	1 318 (e)
Twisted	¾	1 : 2	7 mos.	1 199 (e)
Twisted	¾	1 : 3	7 mos.	701
Twisted	¾	1 : 4	7 mos.	796
Corrugated	¾	Neat cement	7 mos.	962
Corrugated	¾	1 : 1	7 mos.	977
Corrugated	¾	1 : 2	7 mos.	934
Corrugated	¾	1 : 3	7 mos.	735
Corrugated	¾	1 : 4	7 mos.	564
Corrugated	¾	1 : 2 : 4	31 days	640
Thatcher	⅞	....	30 days	646

The following are the authorities for the above tests:

(*a*) A. N. Talbot.
(*b*) C. M. Spofford.
(*c*) New York City Rapid Transit Company.
(*d*) T. L. Condron.
(*e*) Tests of Metals, Watertown Arsenal, 1904.

The Kahn Bar. In the Kahn Trussed Bar (Fig. 9) the attachme stirrups to the tension-member is positively secured. The bars are

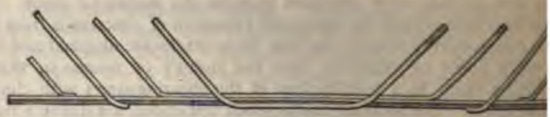


Fig. 9. The Kahn Bar

cross-section with webs rolled on them at two diagonally opposite edg stirrups are formed by shearing these webs through a part of their le turning up parts, as shown in the cut. These stirrups may be placed turn up in pairs or so as to alternate on opposite sides of the bar, ma spacing of the stirrups closer than when turned up in pairs. Another a incidental to the use of this bar is that the greater effective cross-secti steel is at the middle, the point of greatest bending moment with loading. Two disadvantages, however, are the separation of the co the wings above and below the bar, and the limitation as to the effectiv length in deep beams. This bar is controlled by the Trussed Conc Company of Detroit, Mich.

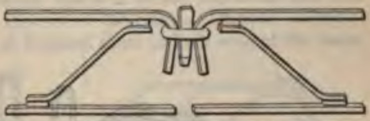


Fig. 10. The Xpantrus Bar

Bar. This bar (Fig. 10) possesses the advantages of rigidly-
mbers, both at the top and at the bottom of beam, providing
iforcement with
over the sup-
ension-member,
r and the top
rm one integral
or mechanical
ling them to-
cting ties being
web of the rolled shape. The original section is retained
ddle of the beam. By slotting the thin webs and after-
the section, the middle web is transformed into a shear-
top flange-bar becomes the top reinforcement. In order
negative bending moment the end of the top bar is bent into
-and-wedge connection is made with the corresponding bar in
n. This bar has been developed into a complete reinforcing-
ll conditions encountered in beam and girder-work. The re-
bricated complete in the shop and is shipped to the job
ssembled and ready to drop into place. Seven sections of bar
manufacturer. Any cross-section can be obtained by merely
sizes. Complete tables in hand-book form are offered by the
use with this system.

ression. The steel reinforcement in reinforced concrete is
ses to assist in developing COMPRESSIVE STRENGTH when the
icient for the purpose, as in the case of beams and girders with
the neutral axis, and columns with rods placed vertically.
el reinforcement in resisting COMPRESSION will be treated more
ivision 3 of this chapter in the paragraph Compression Rods
ders, page 944. On account of the uncertainty, however, of
rete each receiving its proportionate share of the load, the use

Being fabricated off the job there is less interference between fabrication can proceed while the molds are being made, a greater speed in erection is possible. The frames are readily less liable to get mixed than loose rods sent to the job.

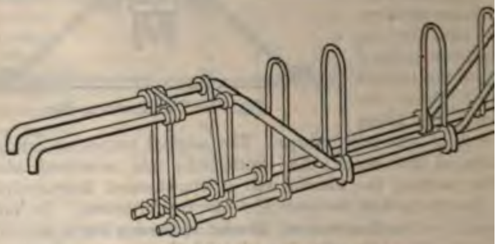


Fig. 11. The Unit System

The Unit System (Fig. 11) is the pioneer of this type of co present is manufactured by the American System of Rei Ill. Its particular features are the bending up of some of the forcements near the supports and the use of round U-shape around the bars while hot and allowed to shrink into place.

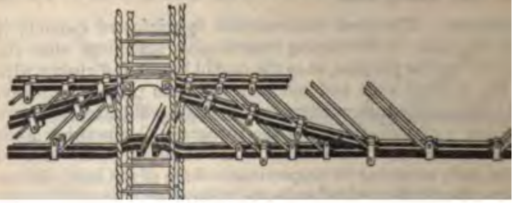


Fig. 13. The Cummings System

Truss. The Luten Truss (Fig. 14) consists of longitudinal rods
members bent diagonally upwards across the beam and con-
ne upper surface to the end of the frame. Diagonal members
rough all the region of diagonal shear in both ends of the beam.

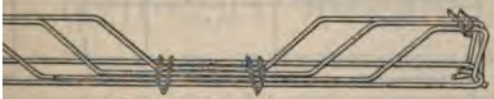


Fig. 14. The Lutin Truss

ovided with a clamp and wedge that locks the members together.
by the National Concrete Company, Indianapolis, Ind.

ar Units. The Corr-Bar Unit, Fig. 15, made by the Corrugated
St. Louis, Mo., is provided with a continuous stirrup of both

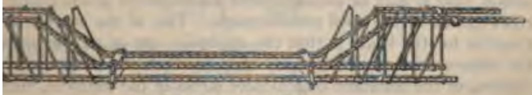


Fig. 15. The Corr-bar Unit

ined web-members with a rigid anchorage at both top and bot-
y Professor Talbot on this type of reinforced beam, considerably
re obtained in vertical shear.

Design of Reinforced-Concrete Construction

ms and Slabs. Different formulas for the design of reinforced
beams, slabs, etc., based on various theoretical considerations,

concrete beam in a state of flexure or bending under a load.

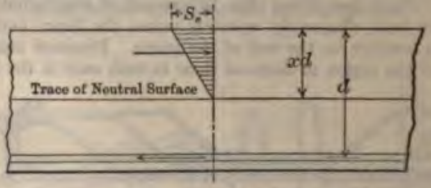


Fig. 16. Sections of Reinforced-concrete Beam

the NEUTRAL SURFACE of the beam or above the NEUTRAL AX
tion are in compression and according to the assumptions t
direct proportion to their distances from the neutral surface c
total area of compression in the concrete, representing the r
STRESS, may be graphically indicated by the shaded triangle.
SIONAL STRESS may be assumed to be concentrated at the cent
cross-section of the steel reinforcement. One of the conditio
LIBRIUM for the beam is that the algebraic sum of all the ho
the cross-section shall be zero; that is, that the sum of a
stresses, or the resultant compressive stress in the concrete, m
or resultant tensional stress in the steel.

Formulas for Reinforced-Concrete Beams. From t
based upon THEORETIC and EXPERIMENTAL LAWS, the follo
derived, in which

S_t = the allowable unit tension or working stress in the s
square inch;
S_c = the allowable unit compression or working-stress in
of the concrete;
r = the ratio of the modulus of elasticity of the steel

of rectangular cross-section

$$M = M_r = Kbd^2 \qquad (1)$$

K being determined by the formula

$$K = S_t \left(\frac{1}{2\left(\frac{S_t}{S_c}\right)\left(1+\frac{S_t}{S_c r}\right)} \right) \left(1 - \frac{1}{3\left(1+\frac{S_t}{S_c r}\right)} \right) \qquad (2)$$

la can be deduced from the LAWS OF FLEXURE of beams and the noted above.

of this formula for the value of K it must be remembered that the S_c for any given ratio of steel to concrete, p, is a constant, so that g values of S_t and S_c must be used. This ratio, p, often spoken of NTAGE OF REINFORCEMENT, is the expression in the first parenthesis member of Formula (2)

$$p = \frac{1}{2\left(\frac{S_t}{S_c}\right)\left(1+\frac{S_t}{S_c r}\right)} \qquad (3)$$

of x is derived from the expression

$$x = rp\left(\sqrt{1+\frac{2}{rp}} - 1\right) \qquad (4)$$

K and x for corresponding values of p, for different conditions building authorities of different cities, are given in Tables V, VI,

ent of resistance" or the "resisting moment" referred to any cross-section horizontal position and in a state of flexure under a load or loads is the f the moments of the internal horizontal stresses with reference to a point and the "bending moment" for that section is the algebraic sum of the the external vertical forces on either side of the section (the forces on the

Table V. Values for Formulas for Reinforced Concrete

$r = 12$

p	x	K	S_c	S_t	K	S_c	S_t	K	S_c
0.0045	0.279	......	...	....	.....	....	....	65.4	516
0.0050	0.291	......	...	....	.....	...	...	72.2	530
0.0055	0.303	......	...	....	69.3	507	14 000	79.2	580
0.0058	0.310	......	...	....	73.0	525	"	83.4	600
0.0060	0.314	......	...	....	75.2	535	"	86.0	612
0.0065	0.325	......	...	....	81.2	560	"	92.8	640
0.0070	0.334	74.7	503	12 000	87.2	587	"	96.5	650
0.0075	0.344	79.5	523	"	93.0	610	"	99.0	"
0.0080	0.353	84.7	544	"	98.8	635	"	101.2	"
0.0085	0.361	89.9	565	"	103.3	650	13 800	103.3	"
0.0090	0.369	94.7	584	"	105.1	"	13 350	105.1	"
0.0095	0.377	99.6	605	"	107.0	"	12 900	107.0	"
0.0100	0.384	104.5	625	"	108.8	"	12 500	108.8	"
0.0105	0.392	109.5	643	"	110.9	"	12 130	110.9	"
0.0110	0.399	112.4	650	11 790	112.4	"	11 790	112.4	"
0.0115	0.405	114.0	"	11 450	114.0	"	11 450	114.0	"
0.0120	0.412	115.7	"	11 160	115.7	"	11 160	115.7	"
0.0125	0.418	117.2	"	10 900	117.2	"	10 900	117.2	"
0.0130	0.424	118.2	"	10 600	118.2	"	10 600	118.2	"
0.0135	0.430	119.8	"	10 350	119.8	"	10 350	119.8	"
0.0140	0.436	121.2	"	10 120	121.2	"	10 100	121.2	"
0.0145	0.441	122.2	"	9 890	122.2	"	9 870	122.2	"
0.0150	0.446	123.2	"	9 660	123.2	"	9 660	123.2	"
0.0155	0.452	124.8	"	9 460	124.8	"	9 460	124.8	"
0.0160	0.457	126.0	"	9 270	126.0	"	9 270	126.0	"
0.0165	0.462	127.0	"	9 100	127.0	"	9 100	127.0	"
0.0170	0.467	128.0	"	8 930	128.0	"	8 930	128.0	"
0.0175	0.471	129.1	"	8 740	129.1	"	8 740	129.1	"
0.0180	0.475	130.1	"	8 580	130.1	"	8 580	130.1	"
0.0185	0.480	131.0	"	8 440	131.0	"	8 440	131.0	"
0.0190	0.485	132.1	"	8 300	132.1	"	8 300	132.1	"
0.0195	0.489	133.0	"	8 150	133.0	"	8 150	133.0	"
0.0200	0.493	134.0	"	8 010	134.0	"	8 010	134.0	"

Table VI. Values for Formulas for Reinforced Concrete

$r = 12$

x	K	S_c	S_t	K	S_c	S_t	K	S_c	S_t
0.217	......	...	....	....	...	......	51.0	506	22 000
0.235	......	...	....	55.3	511	20 000	60.8	562	"
0.251	57.7	503	18 000	64.2	558	"	70.6	614	"
0.266	65.7	542	"	72.9	602	"	78.7	650	21 610
0.279	73.5	581	"	81.6	645	"	82.3	"	20 150
0.291	81.3	618	"	85.4	650	18 910	85.4	"	18 910
0.303	88.5	650	17 900	88.5	"	17 900	88.5	"	17 900
0.314	91.5	"	17 000	91.5	"	17 000	91.5	"	17 000
0.325	94.2	"	16 250	94.2	"	16 250	94.2	"	16 250
0.334	96.5	"	15 510	96.5	"	15 510	96.5	"	15 510
0.344	99.0	"	14 900	99.0	"	14 900	99.0	"	14 900
0.353	101.2	"	14 350	101.2	"	14 350	101.2	"	14 350
0.361	103.3	"	13 800	103.3	"	13 800	103.3	"	13 800
0.369	105.1	"	13 325	105.1	"	13 325	105.1	"	13 325
0.377	107.0	"	12 900	107.0	"	12 900	107.0	"	12 900
0.384	108.8	"	12 480	108.8	"	12 480	108.8	"	12 480
0.392	110.9	"	12 130	110.9	"	12 130	110.9	"	12 130
0.399	112.4	"	11 790	112.4	"	11 790	112.4	"	11 790
0.405	113.9	"	11 450	113.9	"	11 450	113.9	"	11 450
0.412	115.6	"	11 160	115.6	"	11 160	115.6	"	11 160
0.418	117.2	"	10 900	117.2	"	10 900	117.2	"	10 900
0.424	118.4	"	10 600	118.4	"	10 600	118.4	"	10 600
0.430	120.0	"	10 350	120.0	"	10 350	120.0	"	10 350
0.436	121.2	"	10 100	121.2	"	10 100	121.2	"	10 100
0.441	122.2	"	9 870	122.2	"	9 870	122.2	"	9 870
0.446	123.2	"	9 660	123.2	"	9 660	123.2	"	9 660
0.452	124.8	"	9 460	124.8	"	9 460	124.8	"	9 460
0.457	126.0	"	9 270	126.0	"	9 270	126.0	"	9 270
0.462	127.0	"	9 100	127.0	"	9 100	127.0	"	9 100
0.467	128.0	"	8 930	128.0	"	8 930	128.0	"	8 930
0.471	129.1	"	8 740	129.1	"	8 740	129.1	"	8 740
0.475	130.1	"	8 580	130.1	"	8 580	130.1	"	8 580
0.480	131.0	"	8 440	131.0	"	8 440	131.0	"	8 440
0.485	132.1	"	8 300	132.1	"	8 300	132.1	"	8 300
0.489	133.0	"	8 150	133.0	"	8 150	133.0	"	8 150
0.493	134.0	"	8 010	134.0	"	8 010	134.0	"	8 010

0.0075	0.375	......	...	......	92.0	560		105
0.0080	0.384	83.6	500	12 000	97.6	583	"	108
0.0085	0.393	88.6	519	"	103.3	606	"	111
0.0090	0.402	93.5	537	"	109.0	627	"	113
0.0095	0.410	98.4	556	"	114.8	648	"	115
0.0100	0.418	103.3	573	"	117.1	650	13 600	117
0.0105	0.425	108.2	593	"	118.6	"	13 150	118
0.0110	0.433	113.1	611	"	120.5	"	12 760	120
0.0115	0.440	117.9	627	"	122.0	"	12 420	122
0.0120	0.446	122.7	647	"	123.4	"	12 080	123
0.0125	0.453	125.0	650	11 780	125.0	"	11 780	125
0.0130	0.459	126.8	"	11 480	126.8	"	11 480	126
0.0135	0.465	127.7	"	11 200	127.7	"	11 200	127
0.0140	0.471	128.9	"	10 920	128.9	"	10 920	128
0.0145	0.477	130.4	"	10 690	130.4	"	10 690	130
0.0150	0.483	131.7	"	10 465	131.7	"	10 465	131
0.0155	0.488	133.0	"	10 240	133.0	"	10 240	133
0.0160	0.493	133.9	"	10 010	133.9	"	10 010	133
0.0165	0.498	135.2	"	9 810	135.2	"	9 810	135
0.0170	0.503	136.0	"	9 620	136.0	"	9 620	136
0.0175	0.508	137.2	"	9 435	137.2	"	9 435	137
0.0180	0.513	138.2	"	9 260	138.2	"	9 260	138
0.0185	0.518	139.4	"	9 100	139.4	"	9 100	139
0.0190	0.522	140.3	"	8 940	140.3	"	8 940	140
0.0195	0.527	141.1	"	8 790	141.1	"	8 790	141
0.0200	0.531	142.0	"	8 630	142.0	"	8 630	142

VIII. Values for Formulas for Reinforced Concrete

$r = 15$

K	S_c	S_t	K	S_c	S_t	K	S_c	S_t
......	...	....	.....	...	....	60.3	512	22 000
......	...	....	63.5	507	20 000	69.9	557	"
......	...	....	72.3	548	"	79.5	604	"
72.7	528	18 000	80.7	587	"	88.8	646	"
80.5	563	"	89.4	626	"	92.9	650	20 800
88.1	596	"	96.0	650	19 610	96.0	"	19 610
95.6	628	"	99.1	"	18 620	99.1	"	18 620
101.8	650	17 760	101.8	"	17 760	101.8	"	17 760
104.1	"	16 950	104.1	"	16 950	104.1	"	16 950
106.7	"	16 250	106.7	"	16 250	106.7	"	16 250
108.9	"	15 600	108.9	"	15 600	108.9	"	15 600
111.0	"	15 040	111.0	"	15 040	111.0	"	15 040
113.2	"	14 520	113.2	"	14 520	113.2	"	14 520
115.1	"	14 020	115.1	"	14 020	115.1	"	14 020
117.1	"	13 600	117.1	"	13 600	117.1	"	13 600
118.6	"	13 150	118.6	"	13 150	118.6	"	13 150
120.5	"	12 760	120.5	"	12 760	120.5	"	12 760
122.0	"	12 420	122.0	"	12 420	122.0	"	12 420
123.4	"	12 080	123.4	"	12 080	123.4	"	12 080
125.0	"	11 780	125.0	"	11 780	125.0	"	11 780
126.4	"	11 480	126.4	"	11 480	126.4	"	11 480
127.7	"	11 200	127.7	"	11 200	127.7	"	11 200
128.9	"	10 920	128.9	"	10 920	128.9	"	10 920
130.4	"	10 690	130.4	"	10 690	130.4	"	10 690
131.7	"	10 465	131.7	"	10 465	131.7	"	10 465
133.0	"	10 240	133.0	"	10 240	133.0	"	10 240
133.9	"	10 010	133.9	"	10 010	133.9	"	10 010
135.0	"	9 810	135.0	"	9 810	135.0	"	9 810
136.0	"	9 620	136.0	"	9 620	136.0	"	9 620
137.2	"	9 435	137.2	"	9 435	137.2	"	9 435
138.2	"	9 260	138.2	"	9 260	138.2	"	9 260
139.4	"	9 100	139.4	"	9 100	139.4	"	9 100
140.1	"	8 940	140.1	"	8 940	140.1	"	8 940
141.1	"	8 790	141.1	"	8 790	141.1	"	8 790
142.0	"	8 630	142.0	"	8 630	142.0	"	8 630

Cinder Concrete. Values of K for cinder concrete are given in Tabl
and X, which are, however, recommended to be used only for slabs. C
concrete, though an excellent fireproofing material, lacks strength and
be used as a structural material for the slabs, only, between the beams.

Table IX. Values for Formulas for Reinforced Cinder Concrete

$r = 35$

p	x	K	S_c	S_s	K	S_c
0.0005	0.170	7.5	94	16 000	7.5	94
0.0010	0.232	14.8	138	"	14.8	138
0.0015	0.276	21.8	174	"	18.8	150
0.0020	0.311	28.7	206	"	20.9	"
0.0025	0.340	33.9	225	15 300	22.6	"
0.0030	0.365	36.1	"	13 688	24.0	"
0.0035	0.387	37.9	"	12 439	25.3	"
0.0040	0.407	39.6	"	11 447	26.4	"
0.0045	0.425	41.0	"	10 625	27.4	"
0.0050	0.442	42.4	"	9 945	28.3	"
0.0055	0.457	43.6	"	9 348	29.1	"
0.0060	0.471	44.7	"	8 831	29.8	"
0.0065	0.484	45.7	"	8 377	30.4	"
0.0070	0.497	46.7	"	7 988	31.1	"
0.0075	0.508	47.5	"	7 620	31.6	"
0.0080	0.519	48.3	"	7 298	32.2	"
0.0085	0.529	49.0	"	7 001	32.7	"
0.0090	0.539	49.7	"	6 738	33.2	"
0.0095	0.548	50.4	"	6 489	33.6	"
0.0100	0.557	51.0	"	6 266	34.0	"
0.0105	0.565	51.6	"	6 054	34.4	"
0.0110	0.573	52.1	"	5 860	34.8	"
0.0115	0.581	52.7	"	5 684	35.1	"
0.0120	0.588	53.2	"	5 513	35.5	"
0.0125	0.595	53.7	"	5 355	35.8	"
0.0130	0.602	54.1	"	5 210	36.1	"
0.0135	0.608	54.5	"	5 067	36.4	"
0.0140	0.615	55.0	"	4 942	36.7	"
0.0145	0.621	55.4	"	4 818	36.9	"
0.0150	0.626	55.7	"	4 695	37.1	"
0.0155	0.632	56.1	"	4 587	37.4	"
0.0160	0.637	56.4	"	4 479	37.6	"
0.0165	0.643	56.8	"	4 384	37.9	"
0.0170	0.648	57.2	"	4 288	38.1	"
0.0175	0.652	57.4	"	4 191	38.3	"
0.0180	0.657	57.7	"	4 106	38.5	"
0.0185	0.662	58.1	"	4 026	38.7	"
0.0190	0.666	58.3	"	3 943	38.9	"
0.0195	0.671	58.6	"	3 871	39.1	"
0.0200	0.675	58.9	"	3 797	39.2	"

ble X. Values for Formulas for Reinforced Cinder Concrete

$r = 30$

k	K	S_c	S	K	S_c	S	K	S_c	S_t
0.159	7.6	100.6	16 000	7.6	100.6	16 000	6.62	88	14 000
0.216	14.9	148	"	14.9	148	"	13.0	129.5	"
0.259	22.0	185	"	22.0	185	"	19.2	162	"
0.292	28.8	219	"	28.8	219	"	25.2	192	"
0.319	35.8	251	"	35.6	250	15 950	28.5	200	12 750
0.344	42.6	279	"	38.1	"	14 300	30.4	"	11 480
0.365	48.1	300	15 620	40.1	"	13 030	32.0	"	10 420
0.386	50.4	"	14 480	42.1	"	12 060	33.6	"	9 650
0.402	52.2	"	13 400	43.4	"	11 170	34.8	"	8 930
0.418	54.0	"	12 540	45.0	"	10 450	36.0	"	8 360
0.433	55.6	"	11 810	46.3	"	9 860	37.0	"	7 870
0.447	57.0	"	11 180	47.5	"	9 320	38.0	"	7 450
0.460	58.5	"	10 620	48.7	"	8 850	38.9	"	7 080
0.472	59.7	"	10 120	49.7	"	8 440	39.8	"	6 750
0.483	60.7	"	9 660	50.6	"	8 050	40.5	"	6 440
0.494	61.9	"	9 270	51.6	"	7 730	41.3	"	6 170
0.504	63.0	"	8 900	52.5	"	7 420	42.0	"	5 930
0.514	63.9	"	8 560	53.3	"	7 130	42.6	"	5 710
0.523	64.9	"	8 250	54.1	"	6 870	43.2	"	5 500
0.532	65.7	"	7 980	54.7	"	6 650	43.7	"	5 320
0.540	66.4	"	7 710	55.4	"	6 420	44.3	"	5 140
0.547	67.2	"	7 460	55.9	"	6 220	44.7	"	4 970
0.555	67.8	"	7 240	56.5	"	6 040	45.2	"	4 820
0.562	68.5	"	7 020	57.1	"	5 850	45.7	"	4 680
0.569	69.3	"	6 830	57.7	"	5 680	46.2	"	4 550
0.576	69.8	"	6 650	58.2	"	5 540	46.5	"	4 430
0.582	70.4	"	6 460	58.6	"	5 380	46.8	"	4 310
0.588	71.0	"	6 310	59.2	"	5 260	47.3	"	4 210
0.594	71.5	"	6 140	59.5	"	5 120	47.6	"	4 090
0.600	72.0	"	6 000	60.0	"	5 000	48.0	"	4 000
0.606	72.6	"	5 860	60.5	"	4 880	48.4	"	3 910
0.612	73.1	"	5 730	60.9	"	4 780	48.7	"	3 820
0.617	73.6	"	5 610	61.3	"	4 670	49.0	"	3 740
0.622	74.0	"	5 480	61.7	"	4 570	49.4	"	3 660
0.627	74.5	"	5 370	62.0	"	4 470	49.6	"	3 580
0.632	74.9	"	5 270	62.4	"	4 390	49.9	"	3 510
0.636	75.3	"	5 160	62.7	"	4 300	50.2	"	3 440
0.641	75.7	"	5 060	63.1	"	4 220	50.4	"	3 370
0.645	76.0	"	4 960	63.3	"	4 130	50.7	"	3 300
0.649	76.4	"	4 870	63.6	"	4 060	50.8	"	3 240

rced-Concrete Beams of Rectangular Cross-Section. In determin-
E OF BEAM required for any given case, r and the limiting values of
are generally given, and K can be determined for any ratio, p, of con-
eel. The value of M, the MAXIMUM BENDING MOMENT, that is, the
oment at the DANGEROUS SECTION of the beam, is determined from
ions of loading, the span and the spacing; and the width and depth
ms are to be found. Formula (1) may then be put in the more con-
rm.

$$d = \sqrt{\frac{M}{Kb}} \tag{5}$$

A value for *b* is assumed and the equation solved for *d*. Architec structural reasons will often limit the width or depth and several trials to be made.

Reinforced-Concrete Slabs. For the STRENGTH OF SLABS the same apply. A slab may be treated (1) as a rectangular beam of unusu (2) as a series of beams set one alongside the other, the width of each be equal to the spacing of the reinforcing-rods, and one rod being used beam; or (3) as a series of beams of unit width, the area of steel for e being the area of reinforcement per unit of width.

Check-Formulas. It may sometimes happen that it is advisable a given or existing beam-construction as to strength or compliance w fications for working stresses. In that case the following formula convenient (see, also, page 992):

$$M = pS_t bd^2 \left(1 - \frac{x}{3}\right)$$

$$M = \frac{S_c x b d^2}{2} \left(1 - \frac{x}{3}\right)$$

If the strength of the beam for the assumed working stresses is to mined, these values of S_t and S_c are inserted in Formulas (6) and (7 least value of M is used. If the values of M resulting from these eq not equal, the full benefit of one of the materials is not being obtaine stresses in the steel or concrete due to a given loading are to be deter formulas are put in the following forms:

$$S_t = \frac{M}{pbd^2\left(1 - \frac{x}{3}\right)}$$

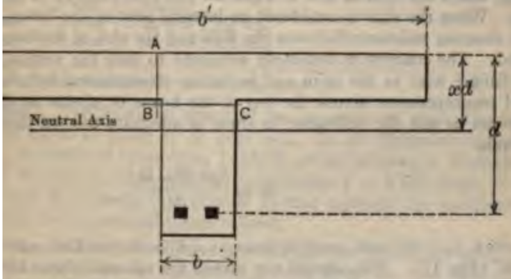
$$S_c = \frac{2M}{xbd^2\left(1 - \frac{x}{3}\right)}$$

These formulas apply to rectangular beams only. M in Formul (9) is the maximum moment due to the external forces, or the maxir ing moment. The value of x can be determined from Tables V to X mula (8) it will be noted that the denominator of the fraction is an for the area of the steel multiplied by the lever-arm of the resisting that is, the distance from the center of gravity of the steel to the cent pression in the concrete. Similarly, in Formula (9), the denomina fraction is an expression for the area of the concrete in compression by the lever-arm, x again being determined by Formula (4) and M maximum bending moment due to the external forces.

Reinforced-Concrete T Beams. Where beams or girders are us forced-concrete building-construction there are usually accompanying If these slabs are cast with the beams or girders they add very m strength of the latter. Some authors prefer not to consider this strength but to regard it as increasing the factor of safety. This posi able only when the slabs are cast or built independently. If made a time, economical design requires that the slab shall be considered. of slab that may be taken as part of the beam should not exceed one span-length of the beam, and the overhanging part on either side

girder construction, the slab action produces compression in the
as the girder-compression with a resulting overstress in the con-
case, transverse reinforcement should be provided at right-angles
d extending well into the slab.

or Reinforced-Concrete T Beams. Fig. 17 shows a cross-sec-
am resulting from the use of the slab as part of the beam, and



ig. 17. Cross-section of Reinforced-concrete T Beam

lso, the notation used in the formulas. In a construction of this
s may be considered:

e neutral axis may fall below the flange, in which case

$$M = S_t pbd\left(d - \frac{t}{2}\right) \qquad (10)$$

$$M = \frac{S_c}{2} b't\left(d - \frac{t}{2}\right) \qquad (11)$$

nulas the small area of concrete in compression below the flange
d the center of compression is assumed to be at the center of
is is done to simplify the formulas. The result is not materially

$$M = \frac{S_c b' t}{2}\left(d - \frac{t}{3}\right)$$

The economical value of p in this case is the same as in Case

Case 3. The neutral axis may fall above the lower edge of t
case is the same as Case 2, since for purposes of calculation all th
flange below the neutral axis is neglected and t becomes xd in tl
last. When the slab is considered an integral part of the beam
and shearing resistance between the slab and the web of the be
vided. The concrete is ordinarily adequate to take the vertic
the flanges next to the stem and is further strengthened by p
steel reinforcements across the top of the beam or girder as
Whether or not the resistance to shear is adequate can be de
formula

$$S_s = \frac{S_h b\,(b' - b)}{2\,t b'}$$

in which S_s is the unit vertical shear at AB, and S_h is the unit
at BC (Fig. 17). This should not exceed the safe unit shear fc
steel reinforcement is provided. The value of S_h in the formul

$$S_h = \frac{V}{b(d - \frac{1}{2}t)}$$

which, it will be noted, is the total vertical shear divided by t
of the stem.

Moduli of Elasticity. In the derivation of all these forn
determination of the values of K, the ratio of the MODULUS c
the steel to that of the concrete plays an important part. It i
to know what values to use. The generally accepted modulu
steel is 30 000 000 lb per sq in. The modulus of elasticity of

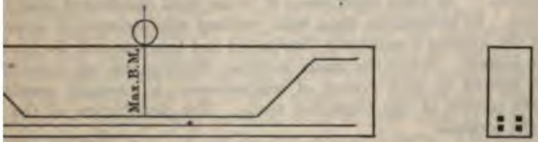
Elastic Properties of Broken-Stone Concrete Twelve-Inch Cubes

tion	Age	Modulus of elasticity in pounds per square inch between loads of			Tests made by
Broken stone		100 and 600 lb per sq in	600 and 1 000 lb per sq in	1 000 and 2 000 lb per sq in	
4	7 days	2 593 000	2 054 000	1 351 000	* Geo. A. Kimball.
4	1 mo	2 662 000	2 445 000	1 462 000	" " "
4	3 mos	3 671 000	3 170 000	2 158 000	" " "
4	6 mos	3 646 000	3 567 000	2 582 000	" " "
6	7 days	1 869 000	1 530 000	....	" " "
6	1 mo	2 438 000	2 135 000	1 219 000	" " "
6	3 mos	2 976 000	2 656 000	1 805 000	" " "
6	6 mos	3 608 000	3 503 000	1 868 000	" " "
12	1 mo	1 376 000	....	....	" " "
12	3 mos	1 642 000	1 364 000	....	" " "
12	6 mos	1 820 000	1 522 000	....	" " "

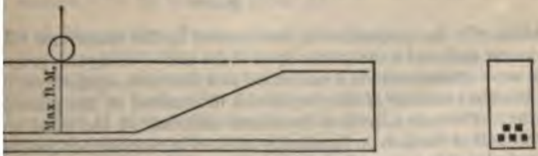
* Tests of metals, U. S. A., 1899, page 741.

Stresses. The WORKING STRESSES for concrete and steel allowed
ies are given in Table II on page 918. In the determination of
of S_c, S_t and r as taken from Table II are substituted in Formula
lue of K may be taken directly from Tables V to VIII, pages 930
bstituted in Formula (5). For M in that formula, the MAXIMUM
ENT due to the external forces is used.

Moments in Beams. Beams and girders are usually considered
AMS, that is, as beams supported at both ends, but not built in,



Reinforcement for Uniformly Distributed or Symmetrically Placed Load



Reinforcement for Unsymmetrically Placed Concentrated Load

continuous, although in many instances they are actually carried,
s BEAMS, over the supports. If continued over a support, there
BENDING MOMENT at that support, and this negative bending mo-
e taken care of by reinforcements in the upper part of the beam.

“That for beams, the bending moment at center and at su spans, be taken at $wl^2/12$, and for end spans it be taken at wl^2, adjoining support, for both dead and live loads.

“In the case of beams and slabs continuous for two spans moment at the central support should be taken as $wl^2/8$ an of the span as $wl^2/10$.”*

Beams simply supported at the ends must be considered with maximum positive bending moments equal to $wl^2/8$. values, w is the load per running foot and l the span in feet.

Bending Moments in Slabs. As floor-slabs are usually ca across the supports, the maximum bending moment due to a uted load is assumed to be less than in beams simply suppo The New York City Regulations provide that “the bendin center and at intermediate supports of floor-slabs continuous supports shall be taken as $Wl/12$.” The same regulations provi ing moments of slabs that are reinforced in both directions four sides and fully reinforced over the supports (the reinforc the adjoining slabs) may be taken as Wl/F for loads in each $F=8$ when the slab is not continuous or when continuous and $F=12$ at both center and supports when the slab is con supports.” In these expressions W is the total distributed l in feet. In rectangular slabs with two-way reinforcement it that the loading is uniformly distributed and that one-half the each system. When the spans in either direction are not eq load carried by each system of reinforcement is inversely p fourth powers of the respective spans. The distribution of by the formula

$$r=\frac{l^4}{l^4+b^4}$$

in which r is the proportion of load carried by the transvers the longer span and b the shorter span of the slab. Using this oading, the full sectional area required must be provided at the point UM BENDING MOMENT. As the supports are approached, part of the nent, as already indicated, is turned up, but care must be taken to keep ributed that at any point there is still sufficient reinforcement below al axis to furnish the necessary tensional resistance. The arrangement cement for a uniformly distributed or symmetrically disposed load is Fig. 18, and for an unsymmetrically placed concentrated load, in Fig. 19. st instance the maximum bending moment is at the middle of the reinforcement is symmetrical about that point, and as much as one-amount of reinforcement may be turned up. In the second instance num bending moment is at some other point than the middle, the rein- must be so disposed that the full amount required will be under the the point of maximum bending moment, and the turning up must etween that point and the support. Other conditions might require half the reinforcement to be turned up. There is another reason for p the reinforcements toward the ends. In addition to the resistance GATIVE BENDING MOMENT, there is a resistance to the SHEAR offered etal running through the concrete at the points where the diagonal ually occur in tests on full-sized beams.

rcentage of Reinforcement. The AMOUNT OF THE REINFORCEMENT e is determined by Formulas (3) and (13) for rectangular and T beams ly. The values obtained by these formulas give the most economi-nt. This may vary from one-fourth of one per cent to one and one-ent of the area of concrete, but will usually run about seven-tenths of ent. The nearest stock size of rods giving this amount or a slightly nount can be selected from the table given on page 1428, or from the s of the manufacturers of the various deformed bars. The NUMBER sed to make up the necessary sectional area must be determined by ions mentioned in the following paragraphs.

umber of Reinforcing-Rods. As already suggested, an even number elf better to a symmetrical or balanced arrangement both in cross-sec-orizontal section. One rod does not permit of the turning up toward the

if, in either direction from that point, the AREA OF CONTA
large enough to make the total ADHESION equal to or greater
If there is a deficiency in this respect it must be made up eit
cal bond or by anchoring the reinforcements at the ends.
ADHESION of concrete and steel are given in Table II, page
to apply, without calculation, to the case of beams with a
moment at the middle is to make the diameter of the rods
two-hundredth of the span. Under ordinary conditions, gene
length of rod on either side of the point of maximum bending
at least eighty diameters for plain rods, and not less than
deformed bars. Under unusual conditions the adhesion sh
studied. The apparent discrepancy between the first and se
this paragraph is explained by an allowance made and based
the tension in the steel does not decrease uniformly with th
tance from the supports. The allowance is purely arbitrary
safe. For cases of unsymmetrically loaded beams it is best
fully into the conditions.

The Separation of the Rods. It has not been unusual
to have the concrete split off from the under side along the lin
ment. This is due in part, if not entirely, to an insufficiency o
and around the reinforcement. To avoid this the SPACING
the reinforcing-rods in the cross-sections of the beams must
resistance of the concrete to SHEAR at the level of the rods is
the ADHESION of the concrete to the steel. As a general rule
spaced not less than two and one-half diameters on centers
ameters from the sides of beams. The clear distance between
between rods and edges of beams should in no case be less
formed bars, if stressed to their full tensional value, should
apart than plain bars. At the middle of a beam, the BOND-ST
the top of a continuous beam, over the supports, where the ne
creases rapidly, the bond-stress is apt to be excessive and fre
diameter of the reinforcement.

When the resistance of the concrete to shear is not sufficient, web-
nt must be provided by one of the following methods or by a com-
them:

ttaching to or looping around the horizontal members, stirrups or
rtical members;
ecurely attaching inclined rods to the horizontals in such manner as
prevent slipping;
ending of a part of the longitudinal reinforcement at certain points,
us providing against the diagonal tension and allowing a sufficient
ount of horizontal steel to remain to resist the direct tension.

omary to use the calculated VERTICAL SHEARING-STRESS as a measure
ONAL TENSILE or WEB-STRESSES. In all cases, the concrete may be
carry its safe load, and it is ordinarily assumed that two-thirds of
AL VERTICAL SHEAR is resisted by the web-reinforcement. For beams
with web-members, the total VERTICAL EXTERNAL SHEAR over the ef-
ion should not exceed 6% of the COMPRESSIVE STRENGTH of the con-
Regulations of the Bureau of Buildings of New York City specify
EARING-STRESS in concrete, when all the DIAGONAL TENSION is resisted
hall not exceed 150 lb per sq in. For beams in which part of the
l reinforcement is in the form of bent-up rods, the MAXIMUM VERTICAL
TRESS should not exceed 3% of the COMPRESSIVE STRENGTH of the

sses in web-reinforcements may be determined by the following

$$P = Vs/l \qquad (19)$$

s inclined 45°, not bent-up bars,

$$P = 0.7\, Vs/l \qquad (20)$$

is the horizontal spacing of the web-members, V the total external
ar, l the effective depth from center of compression to center of steel

some distance.

Attached Shear-Members. Stirrups need not be firmly att
sional reinforcement; but the allowable BOND-STRESSES and SH
in the concrete must not be exceeded in transmitting the stresses
and longitudinal rods. The stirrups and inclined members m
sufficient BOND-STRESSES to transmit the entire stresses for wh
signed, and they must sometimes be supplemented with anchor
pression-side of the beam. It is, perhaps, better to have th
they will certainly assist in anchoring the tensional reinforce
forms of stirrups and methods of attachment are used. In t
(Fig. 9) and the Xpantrus (Fig. 10) the stirrups form a part
reinforcement. The U form, either upright or inverted, is a ve
of stirrup, and may be a rod of either round or square cross-
strap as shown in Figs. 8 and 13. The Hennebique system c
clined rods and vertical stirrups. In some cases, when the slal
constructed together, the slab-reinforcement is carried through
of the stirrups.

The Bond between Steel and Concrete. The BOND betw
tension and the concrete must not exceed the safe working val
is not sufficient, the rod will slip. Tension-rods must, therefo
large to develop sufficient BOND-STRENGTH to transmit the
bent-up bars are employed, the BOND-STRESSES in places, in
and bent bars, will be higher than if all bars were straight. In
the ends of the bars at the supports are fully stressed and t
carried into the supports and anchored to develop this stress
bars, an additional length must always be provided above that
assumption of UNIFORM BOND-STRESSES. Wherever possible,
strength should be provided throughout the length of the bar
end-anchorage. Between plain bars and concrete the BOND-S
assumed to be 4% of the COMPRESSIVE STRENGTH of the concrete

**The Breadth of a Reinforced-Concrete Beam of Rect
Section.** The breadth of a rectangular beam, and of the ste

s: (1) concrete with VERTICAL REINFORCEMENT near the outer) concrete wrapped with SPIRALLY-WOUND WIRE or with metal oncrete with a METAL CORE.

f Columns. The lengths of reinforced-concrete columns are vari- by different authorities as follows, the figures being in each case THE LENGTH to the LEAST LATERAL DIMENSION:

rk	15
	12
phia	15
s	15
d	12
re	16
ncisco	15
	16
	12

limits, also, the least side or diameter to 12 in and San Francisco

-Reinforced Columns. In determining the strength of columns L REINFORCEMENT, the steel is assumed to carry a load per square the working load per square inch on the concrete times the ratio of elasticity of the steel and concrete. The allowable stresses, ratio of are given in Table II, page 960. For example, in New York a load sq in is allowed on the concrete, and 12 times 500 equals 6 000 lb he steel, 12 being the ratio of the moduli as fixed by the regulations. 1 nor more than 4% of vertical reinforcement should be used in ncrete columns. The reinforcing-rods should be tied together at intervals of not more than the least side or diameter of the column. s, to a great extent, the buckling of the reinforcement under load equent splitting of the concrete. The VERTICAL REINFORCEMENT, erve its purpose of taking up the bending in the column, should be ar the outer surfaces of the column as possible, consistent with ction of the steel. (See page 959.) If tension is possible in the

COMPRESSIVE STRESSES developed by the restraining action
bands at right-angles to the direct compressive stresses. B
elasticity, however, very little stress is developed in the lat
tests show that at an early stage, the deformation or shorten
is equal to that of plain concrete. With further loading, the
work and prevent failure, thus increasing the so-called TOUGHN
and the ultimate compressive or breaking strength. This ef
riously allowed for by considering the hooping-metal equivale
by imaginary longitudinals. Considère and other investigator
the hooping is equivalent to 2.4 times as much longitudinal s
ally conceded that hooping permits of a somewhat higher unit
crete. The regulations of New York City provide that "axi
columns with not less than one per cent of hoops or spirals s
apart than one-sixth of the diameter of the enclosed column, a
than three inches, and with not less than one nor more than fo
tical reinforcement, shall not exceed 725 pounds per square inc
within the hoops or spirals nor 8 700 pounds per square inch
inforcement." St. Louis and Cleveland permit 2.4 times the
to be considered as longitudinal reinforcement; Chicago 2.5 t
nati 2.2 times.

The Considère Formula. In the following formula the
to embody the results of the investigations of Considère a
relation of the INCREASE IN COMPRESSIVE STRENGTH to the am

$$P = 1.5\, S_c \pi r^2 + S_t \frac{A_h}{p} \pi^2 r + 1.5\, n S_c A_s$$

in which

P = the safe load in pounds;
S_c = the safe unit working stress of ordinary reinforced
pression;
S_t = the safe unit tensile stress in the hoops;
r = the radius of the hoops or wrapping surrounding th
A_h = the area of the cross-section of one hoop;

lumns, thus affording good lateral support, equal to the
e better to omit the wrapping and avoid the possible com-
inforcement from column, girder and floor-construction and
king of the bond of the concrete. The materials used for the
steel wire or steel bands. When wire is used it is spirally
ous through the full length of the column. The ends of the
the column and turned down to such an extent that when
en poured and set, there will be sufficient anchorage to resist
rapping due to the outward pressure of the concrete. When
ed, as in the Cummings system, care must be taken to make
n the bands as strong as the bands themselves. A form of
the merits of rapidity and ease of erection is shown in the
he Bush Terminal Warehouse, Borough of Brooklyn, New
d on page 959.

lumns. The object of this type of column is to provide a
ll or heavily loaded buildings that will have the necessary
ot encroach too seriously on the floor-space. For this form
gineers advocate placing a steel core through the axis of the
aking the bulk of, if not the entire, load.* "A rational basis
mine the strength of the steel column by the use of the column-
per l/r of the column and to consider the concrete of the core-
ress-value proportional to the strength of plain concrete." †
designed a column (Fig. 20) for the McGraw Building, New
teel core has sufficient strength as a column, independent of
rry the entire dead load coming upon it, the stresses in the
ase greater than those allowed on steel columns under the
Code, considering the ratio of length to radius of gyration.
stresses were not allowed to exceed 9 000 lb per sq in in any
ds were provided for by placing enough concrete within the
prevent the stress on the concrete from exceeding 750 lb
s one-twelfth of the maximum allowable load on the steel.
de of the steel was considered only as a protection against fire
lumns of this type should be designed with caution. The

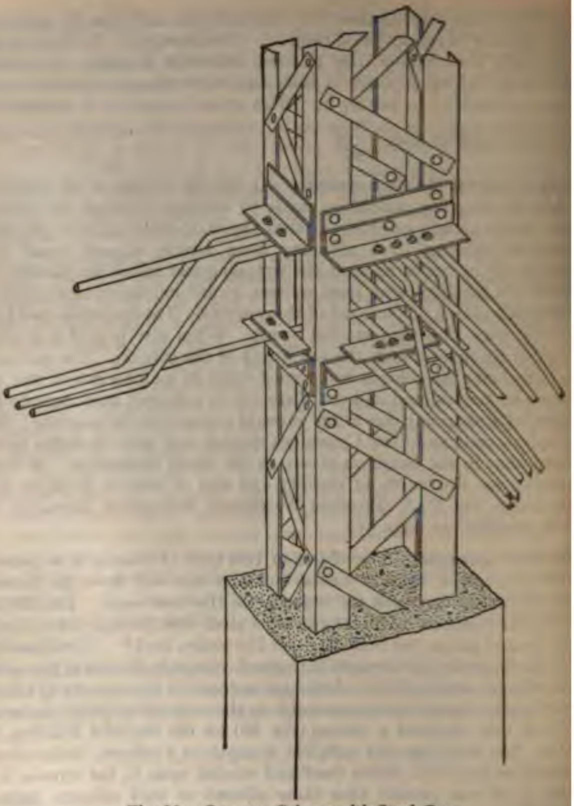
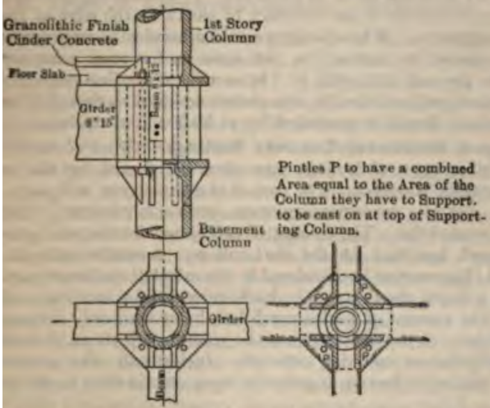


Fig. 20. Concrete Column with Steel Core

Rich Mixtures of Concrete. Increasing the proportion of cement
mixture increases the ultimate strength of the concrete proportionally a
effective in designing columns with smaller cross-sectional area. The incr
compressive strength is also accompanied by a higher modulus of ela
Furthermore, the employment of a rich mixture also permits of higher pr
tional stresses in the steel and consequently a more economical design.
internal stresses in a monolithic member, however, may be considerably
plicated by the excessive shrinkage of rich mixtures which have a tenden
crack. The New York City Regulations provide that "in reinforced-concre
umns the compression on the concrete may be increased twenty per cent
the fine and coarse aggregates are carefully selected and the proportion of ce
to total aggregate is increased to one part of cement to not more than fou
one-half parts of aggregate, fine and coarse, either in the proportion of
part of cement, one and one-half parts of sand and three parts of stone or g
or in such proportion as will secure the maximum density."

arings for the concrete construction and so that there will be a CON- that construction; for the great advantage in reinforced-concrete n lies in its MONOLITHIC character. When cast-iron columns are eads of the columns may be cast with openings through which the



Pintles P to have a combined Area equal to the Area of the Column they have to Support. to be cast on at top of Supporting Column.

Connections for Cast-iron Columns and Reinforced-concrete Construction

nt may pass from one side to the other. Fig. 21 shows how this has n a building at Gay and Christopher Streets, New York City, withng the strength of the columns at the connections.

olumns. In STEEL COLUMNS it is simpler to provide connections e reinforcing-rods and the steel shapes of the columns. When the nt does not go through the columns, some rods should be placed them to tie as much as possible the concrete on one side to that on

stances they are not as economical as brick walls. If reinforc
bearing-walls, they can be reduced to about two-thirds the thi
walls, provided, however, that the load on the concrete does not
load per square inch permitted on reinforced columns. The ratio
height to thickness should not exceed that fixed for columns.
walls, supported entirely on girders, the minimum thickness
Such walls should be reinforced with not less than ½ lb of steel
of wall, in the form of rods placed vertically and, less frequent

Reinforced-Concrete Footings. (See, also, pages 186, 225
principles underlying the design and construction of reinforced-
are the same as those applied to other types of footings.
column-footings the overhang or off-set must be considered
CANTILEVER loaded uniformly with a load per square foot equal
square foot imposed on the underlying soil. The reinforcing-ro
essarily be placed near the lower surface of the footing and the
determined by formulas given on page 929. A detail often ov
forced-concrete footings is the tendency to SHEAR at the edge
or column supported. When footings would otherwise become
cantilevers should be resorted to, the same as for steel con
pages 165 to 169 and 978 to 982.) The maximum bending mo
tilever is determined and the concrete girders designed as descri
Steel in footings should be protected by at least 4 in of concre

Economy of Reinforced-Concrete Footings. Great econ
grillage or other types of footings may often be effected by t
FORCED-CONCRETE FOOTINGS. The cost of the latter type will
40% of the cost of a corresponding STEEL-GRILLAGE FOOTING.
is easily accounted for. The amount of excavation for the rein
generally much less than for the steel grillage. A smaller am
is used, and this concrete is considered in the calculations for st
in the steel grillage, the concrete is chiefly provided for incasin
the steel. The amount of steel is much less, being used only to
sional resistance of the construction, the compressive strength
by the much cheaper material, concrete. Incidentally, the p
steel in the reinforced footing is generally more certain than in t
footing.

Concrete Piles. Concrete piles are discussed in Chapter I
200 and some of the types are there described.

Connections in Reinforced-Concrete Construction. Much
can be displayed and must be exercised in the design of the
connections. The great value of reinforced-concrete construc
types is the possibility of securing great RIGIDITY. This can
when the result is as nearly MONOLITHIC as possible. We the
take up vibration and this advantage, in the case of workshops

a test of their strength was made, without distress, by dropping a four bags of cement, weighing about 380 lb, from the floor above to ediate platform, a distance of 11 ft. No injurious effects were noticed.†

4. Types of Reinforced-Concrete‡ Construction

onstruction. In localities where the cost of labor is high and where tions cause more or less congestion, it is probably more economical to instead of concrete for the walls. In such cases the type of construc- ilar to ordinary MILL-CONSTRUCTION. Provision must be made to an- eams and girders, and this can be done by bending the ends of the -rods so that they will extend horizontally into the walls on each side.

n Construction. The SKELETON TYPE of construction seems to be est adapted to reinforced concrete. A framework of columns, girders, flooring is built, as in steel construction, the wall-girders and columns, being designed to carry the weights of the outside walls as well as of the floor-loads and live loads which comes on them. The work, in of construction, can generally progress more steadily than in the MILL- TION since the concrete work need not be stopped at any time to wait ickwork to be carried up, if brick is used for the walls. In the SKELE- TRUCTION any type of outside wall may be used; brick, concrete, tile, ome cases the panels are simply filled in with brickwork, 8 or 12 in ving the concrete columns and girders showing between the brick For walls situated on property-lines where adjoining buildings are likely ted, this is not objectionable. If the wall remains exposed and a good e is a consideration, the columns and girders can be treated architec- set off the brickwork; or the brickwork may be continued as a facing utside of the columns and girders. This was done in the Bush Termi- ouses, Borough of Brooklyn, New York City.§ To thoroughly secure facing, galvanized anchors were placed in the concrete columns and they were erected, projected sufficiently to bond into the brick joints. oncrete for the panels the sides of the columns are cast with pockets, r recesses to receive the panels, which, as in the case of brickwork, are

affords good insulation. The particular advantage in the s
construction, especially for workshops and factories, is the p
window-areas affording light and ventilation.

System M. A type of construction known as System M h
by the Standard Concrete Company of New York City (Fig.

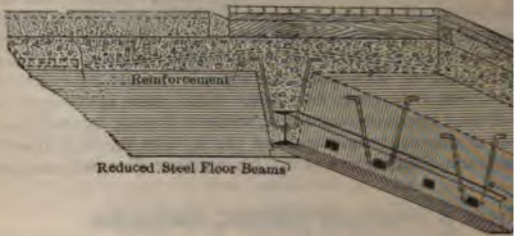


Fig. 22. System M Type of Reinforced-concrete Constr

of a light steel skeleton frame designed to carry the dead l
structure, except that the columns are designed to carry the
structure is incased in concrete making ultimately a reinforced-
tion.* Its advantage consists in its adaptation to the erection
the steel reinforcement before even the centers or molds are p
Under congested conditions, such as prevail in large cities, it i
construction. The use of the steel in this type is, however,
In order to get the necessary strength in the steel framework
used which do not offer the amount of adhesion that should
amount of metal used. Furthermore such shapes must necess
to some bending which tends to break the bond between c

Flat-Slab Construction.† In this form of construction b
are eliminated, almost completely if not entirely, and the sla
directly on the columns; the tops of the columns are enlarged in
This system of construction employs a shallower floor-construc
narily attainable. The floor-centering, too, for purposes of ere
simpler, especially in those forms of slabs in which the lower s
plane. "At present, a considerable difference of opinion exists
as to the formulæ and constants which should be used, bu
tests are accumulating data which it is hoped will in the n
the formulation of the principles of design for this form of
There are in use at present four principal types, on all of wh
are claimed. In two of these the reinforcement, spoken of as
FORCEMENT, is placed and run from column to column, parallel
the slabs and diagonally across each slab. The other two ty

ever Flat Slab, controlled by the Concrete Products Company,
is the second of the FOUR-WAY REINFORCEMENT-SYSTEMS. It
he one described in the preceding paragraph mainly in the con-
e column-cap. The column-bars are not bent to the shape of the
ue up straight. The horizontal cap-reinforcement is provided by
rame of radial bars, held together by a Diamond Bar which is in-
st the circumferential stresses. The diameter of the cap is about
and the thickness of the slab about 1/25 the span. The two
NFORCEMENT-SYSTEMS are the Akme System, controlled by T. A.
cago, Ill., and the Corr-Plate Floor System, controlled by the Cor-
Company, New York City and Buffalo, N. Y. The thickness of
lly about 1/25 the span. Whenever necessary to provide for large
ses and bending-stresses around the column the slab is increased in
at point, forming, in appearance, an extended cap at the column-

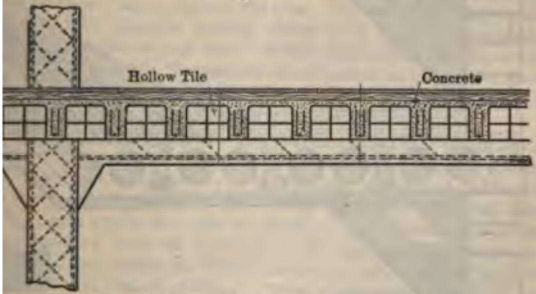


Fig. 23. The Kahn Tile and Reinforced-concrete System

Akme System the reinforcement is concentrated over the column-
practically in broad, flat girders of the width of the caps, the

inches wider than the concrete beams, are placed under the spaces
by the beams, and the tiles are laid in rows and supported along their
the planks, thus forming the sides of the molds for the beams. The
ment is placed and the concrete poured, with or without floor-plates, as
sities of the case may require. Care must be taken in pouring the
that the tiles are not displaced sidewise. The tiles should fit closely
joints, otherwise the finer particles of the concrete are liable to flow
either making the concrete porous or requiring more cement and
necessary. This form of construction, besides being economical in
offers the advantages of a flat ceiling without the application of
roof-construction particularly, of freedom from condensation.

The Floretyle Systems. A floor-construction similar to the
construction just described has been devised by the Trussed Conc

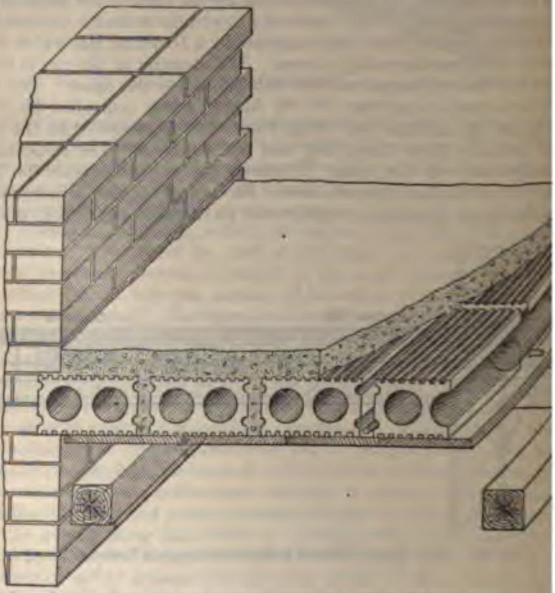


Fig. 24. The Faber Two-way Tile and Reinforced-concrete System

Company of Detroit, Mich., in which steel forms, called Floretyles,
bottom blocks. The advantages claimed over the terra-cotta tile
lighter weight of construction, larger covering capacity and greater
centering. The Floretyles are furnished in lengths of 3 and 4 ft and
of 6, 8, 10 and 12 in. The width at the base is, in all cases, 21 in, wit
tapering at an angle of 7° 30′. They are furnished in two types,
serrated edges for use with the company's Hy-rib lath for ceilings, or wi
edges for use where paneled ceilings are required.

Two-Way Tile Systems. The same principle of construction
systems just described is involved in the Faber Construction (Fig. 24)

em B, using the standard floor-tiles with rectangular spacing of s; and System C, combining standard floor-blocks and special angle-he Floredome construction, put on the market by the Trussed Concrete pany, Detroit, Mich., the tile spacing-blocks are replaced by rectan-e-shaped steel forms with the under side open. Lightness in floor-se and rapidity of installation and no breakages are the advantages The ceiling-treatment in this construction is similar to that in the ystem. The base of the domes is uniformly 21.5 in square; the depth g 6, 8, 10, or 12 in.

h of Combination-Systems. While the tiles may under favorable add to the strength of the combined floor-construction, the chances ctory workmanship are too great to consider them in the calculations h. In the floors reinforced in one direction, the construction should as a series of either rectangular beams or T beams, as the concrete her to or above the top face of the tiles. The TWO-WAY REINFORCED ION should be similarly treated, either as a series of intersecting s a slab supported on four sides. If the construction is to be treated f T beams or as a slab, the concrete should extend at least 2½ in above face of the slab and the tiles or fillers should not exceed 60% of the the construction.

ely-Molded Construction. The UNIT OR SEPARATELY-MOLDED CON- consists of precast reinforced-concrete members, columns, girders, labs, either molded at the site of the building or made at the factory d to the site ready for use. The various systems of fire-proof floor-ibed on pages 859 to 862, may be combined with columns and girders construction to form the entire structure, each member being cast und, swung into place and fitted together in the structure by inter-nforcement and poured grouting. Great economy is claimed for this erection on account of the saving of forms. The disadvantage of tem, however, appears to lie in the lack of sufficient rigidity in tall molded unit structures. All floor-members must be designed and nple, NON-CONTINUOUS UNITS with the reinforcement left projecting ds to serve for tying the structure together. These junctures are

West. (See page 861.)

5. Fire-Resistance of Reinforced-Concrete Co

Non-Conductivity of Reinforced Concrete. Concrete of heat and in this fact lies whatever virtue it has as a fire- series of tests made by Professor Woolson of Columbia Un City, and reported at the 1907 meeting of the Amercian Societ rials, shows the following results:

(1) "That all concrete mixtures when heated throughou of 1 000° to 1 500° F. will lose a large proportion of their str and that this fact must be well remembered in designing."

(2) "That all concretes have a very low thermal condu lies their well known heat resisting properties."

(3) "That as a result of this low thermal conductivity, t half inches of concrete covering will protect reinforcing m heat for the period of any ordinary conflagration (provided concrete stays in place during the fire)."

(4) "That reinforcing metal exposed to the fire will not tivity an injurious amount of heat to the embedded portion

(5) "That the gravel concrete was not a reliable or safe gate." †

Loss of Strength of Reinforced Concrete. If its NON all that is involved in the fire-proof character of concrete, ness required for the protection of the steel could be easil the STRENGTH of the concrete is more or less affected when heat. An effort has been made to determine this effect and results as reported by Professor Woolson of Columbia Un City, is given in Tables XII and XIII.

Fire-Tests on Reinforced Concrete. The EFFECT OF concrete has been studied in a number of tests made by the of New York City and Philadelphia, and in some of the rec

nineteen tests made under the auspices stated, seven were fail-
passed, showing more or less satisfactory conditions.*

XII. Tests of Concrete Blocks Heated on All Sides †

ecimens, 6 by 6 by 14-in prisms; proportions 1 : 2 : 4
Age 2 months; temperature 1500° F.

ment	Aggregate			
	Limestone	Trap-rock	Cinder	Gravel
ticity,				
sq in:				
....	6 000 000	3 430 000	1 090 000	8 000 000
ours............	200 000	150 000	49 500	....
ours............	....	129 000	571 000	....
sq in:				
....	6 000 000	4 355 000	960 000	6 887 000
ours............	285 000	222 000	....	....
ours............	....	188 000	....	....
sq in:				
....	5 647 000	4 355 000	915 000	6 000 000
ours............	425 000	348 000	....	....
n lb per sq in:				
....	2 740	3 140	1 400	2 780
urs............	1 345	1 400	547	....
urs............	870	997	504	....

XIII. Concrete Blocks Heated on One Face Only ‡

ecimens, 6 by 6 by 14-in prisms; proportions 1 : 2 : 4
Age 2 Months; temperature 1 500° F.

Treatment	Aggregate	
	Limestone	Trap-rock
ticity, heated 5 hours:		
sq in........................	293 400	200 000
sq in........................	521 700	268 000
sq in........................	730 700	379 000
n lb per sq in...............	1 840	1 705

n, from a study of the tests in detail,§ shows that to a depth 1 in the concrete is seriously impaired and easily washed off by a lied to the surface. Any stone containing an appreciable per- onate of lime will calcine and cause failure. Where the con-

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list of these tests, see Table in Proc. Am. Soc. for Test. Mats., Vol. VI, l tests have been made since that report was submitted.

c. Test. Mats., Vol. VI, page 446.

c. for Test. Mats., Vol. VI, page 448.

reports are on file in the Bureau of Buildings, Borough of Manhattan,

the fall of a heavy tank that had been supported by the
time an adjoining building constructed with unprotected st
was twisted into a tangled mass of metal.

Tests in the Baltimore Fire. In the Baltimore fire th
inforced-concrete building of the three exposed to the fire,
conclusion can be drawn. In one of the buildings, the c
was entirely destroyed, but this was probably due to the f
failure of other non-fire-proof parts. In a second buildi
forced-concrete floor of a banking-room came out practicall
was not exposed to severe fire. The third structure was,
severe fire. The contents of the building were destroyed an
outside brick walls fell. The floors, five in number, were
crete supported on concrete columns, having replaced an o
struction. A test made after the fire showed that the flo
enough to sustain the loads for which they were designed,
slabs were cracked. The girders were cracked longitudinal
the reinforcement, and the columns were spalled to such an
most of the reinforcement. It would have been difficult to
so that it would resist another such attack.*

Tests in the San Francisco Fire. The EFFECTS OF T
construction in the conflagration immediately following the
quake in 1906 are summed up in the following paragraph
committee of engineers that investigated the subject.†

"Concrete floors generally had hung ceilings, and, where
uninjured. Where exposed, the concrete is in most cases de
in the Sloan, Rialto, and the Aronson Buildings, and the
The concrete is dry, and while in many cases hard, yet all
burned out and it may be said to be destroyed, even if able
Floor coverings of wood invariably burned, adding to the de
were generally burned. Surfaces of cement mortar fared mu
leum covering remaining practically intact." ‡

In discussing the report, Mr. A. L. A. Himmelwright, w
inspection of the ruins concludes that reinforced concrete

rwriters' Requirements for Reinforced Concrete. The follow-
† are those given in the Building Code recommended by the
rd of Fire Underwriters. They are generally considered, except
rs and some fire-engineers, as ultraconservative.
num thickness of concrete surrounding and reinforcing members
ch or less in diameter shall be one inch; and for members heavier
rter inch the minimum thickness of protecting concrete shall be
taking that diameter, in the event of bars of other than circular
which lies in the direction in which the thickness of the concrete
but no protecting concrete need be more than four inches thick
size; and provided, further, that all columns and girders of rein-
e shall have at least one inch of material on all exposed surfaces
e that required for structural purposes; and all beams and floor
ve at least three-quarters inch of such surplus material for fire-
oses."

ms of Protection for Reinforced Concrete. Because of the
ed by fire on reinforced concrete, as above described, and the
estoring the construction where so affected, various suggestions
de to protect the concrete construction with other materials. On
s excellent fire-resisting qualities (see page 818), CINDER CON-
lly suggests itself. This material is out of the question where
quired. But its use may be combined with that of STONE CON-
cing a sufficient thickness for protective purposes on the outside
ements in columns, below the neutral axis in beams and girders ‡
der surface of floor-slabs. Difficulties are likely to be encountered,
acing two kinds of concrete in the same mold, but these difficulties
nountable. Careful inspection is required to see that the poorer
put in place of the stronger. One kind of concrete should follow
ediately in order to secure a bond between the two. This sug-
atisfactorily applied to the column-protection in the Bush Ter-
uses in the Borough of Brooklyn, New York, serving at the same
purpose. The steel-wire wrapping for the columns was prepared
ft in height. Metal lath with about a ½-in mesh was placed

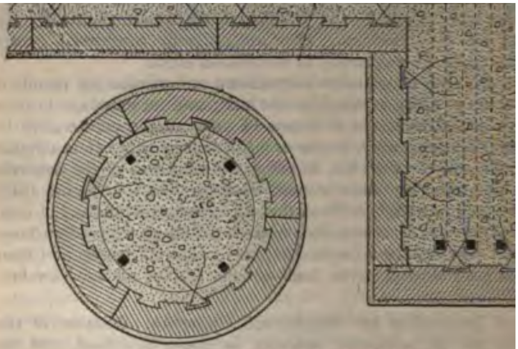


Fig. 25. Tile Protection for Reinforced Concret

completely incased with HOLLOW-TILE BLOCKS. Being either or forming them, their rough and furrowed porous surfaces ca firmly to the concrete. They afford as efficient protection l steel columns, and if destroyed the blocks can be replaced.

6. Protection Against Corrosion in Reinforced-Concr

Thickness of Reinforced Concrete. The THICKNESS OF for protection against fire has been found to be also ample for CORROSION. It is well established that steel embedded in ne corrode. C. L. Norton of the Massachusetts Institute of T Mass., draws the following conclusions from a series of ex 1902 and 1903.*

(1) Steel embedded in neat cement is secure against corros

(2) Steel embedded in a dense concrete mixture is safe aga

n account of the fine cracks due to tension in beams and girders.

Steel in Cinder Concrete. Cases are on record of serious TEEL embedded in CINDER CONCRETE. In a report to the Struc- a of San Francisco, Cal.,† the committee investigating the sub- in cinder concrete "the extent of the corrosion is great enough langer the safety of the floors, and it is not probable that the ve supported their loads more than one to three years longer." recommended "that the Structural Association try to amend lding law so as to exclude the use of cinder concrete in floor- proofing."

H. Fox in his investigations ‡ on this same subject finds that rty days' treatment, the specimens were broken, and the steel ned for corrosion. With but one exception, one or more of the es in each specimen showed unmistakable signs of corrosion. ade no difference how the concrete was mixed, wet or dry, tamped whether the steam or water treatment was used, the result was streaks and spots were found; the difference in the amount of imperceptible." He concludes that "to secure a dense homo- concrete, a thorough tamping is necessary. A rich mixture, one in which the proportion of cement to aggregate is larger, in all cases. The greatest of care should be taken in mixing the may be necessary to resort to the seemingly impractical method einforcement with grout before placing in the concrete."

chemical and physical tests,§ made by George Borrowman of of Nebraska, it was found that disintegration of cinder concrete the oxidation of iron and sulphur producing internal stresses cracking with occasional efflorescence of ferrous sulphate on the these tests, it was concluded that cinders with much sulphide lphur are likely to give unsatisfactory results, especially if there porous material present; also that such material (cinders) may allowed to weather with occasional washing, until the ferrous ur have been washed and leached out of the cinders. The these tests were from carefully screened steam coal and slack. owed considerable ferrous iron and sulphur as sulphide and sul-

ion of the CORROSION OF STEEL in CINDER CONCRETE Professor es: "There is one limitation to the whole question, that is, the etting the steel properly incased in concrete. Many engineers g to do with concrete because of the difficulty in getting 'sound'

Turneaure in Trans. Am. Soc. for Test. Mats., Vol. IV, page 505.
ng News, Nov. 1, 1906, page 458.
ng News, May 23, 1907, page 569.
Industrial and Engineering Chemistry, *June, 1912.*

by thorough wet mixing, causes the rusting of any steel which i
is one cure and only one, mix wet and mix well. With this p
trust cinder concrete quite as quickly as stone concrete in t
rosion."*

In 1902 the Pabst Building in New York City, an eight-st
construction, was taken down after standing for about four
filling between the steel I beams in this case consisted of c
metal lath, built in segmental form. The Roebling const
The steelwork generally was found to be free from rust, th
remembered that all the steel had been painted.† Taking a
sideration it is probably safe to use cinder concrete, if care i
a proper mixture and careful and thorough workmanship.

7. Erection of Reinforced-Concrete Constr

Forms for Reinforced Concrete. For the erection of r
it is generally necessary, first, to construct MOLDS or CENT
umns, floors, etc. Wood is the material used for this pur
centering has been used with questionable success and econo
tion of the wood for the molds a clean grade of dressed pi
It should be thick enough to resist warping and to resist defle
ports. It must be coated on its surface with soap or some
substance to prevent it from sticking to the concrete. The fo
be erected carefully, the exact size of the proposed parts an
position and direction. For floor-molds, sufficient supports
not only to carry safely the heavy wet concrete, but also su
liable to be placed on the floors up to the time when the con
ciently to carry such loads. The supports must have su
prevent deflection in the molds. The molds should be so con
can be easily removed when the concrete has set. Sharp
avoided as much as possible, as the wood is liable to stick
there are reentrant angles in the finished concrete work, the
beveled edges, and at salient edges of the finished concret

dings forms a
art of the cost
ı. Economy in
n be affected in
l planning by
floor-panels
building uniform
repeating, as far
ch parts as piers,
ccessful attempts
ade to dispense
tion of timber
ntering by cast-
us members of
n on the ground
g and erecting
same way that
columns, beams
assembled and
page 955.)

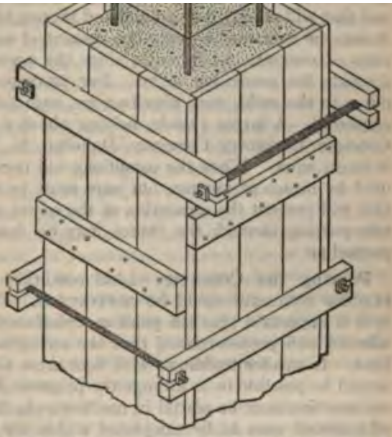


Fig. 26. Wooden Form for Reinforced-concrete Column

Iixing. In all
crete work the
ıld be MIXED
. Satisfactory HAND-MIXING can be obtained and might be
very small jobs, where it would be uneconomical to set up
XER. But a much more uniform product will result from machine-
ost types of mixers are mounted on wheels so as to be easily moved
chanical mixers are either CONTINUOUS MIXERS or BATCH-MIXERS.
ous mixers the materials are fed sometimes by hand and sometimes
and the concrete issues continuously. The product, however, is
e as uniform as that from the batch-mixer; for when the latter
der constant supervision, whereas when the continuous mixer is
ine is relied upon. Of the batch-mixers the ROTARY TYPE is the

and that will readily flow around and thoroughly incase all steel
It may be necessary to vary the amount of water somewhat in
mass of concrete, as in walls, since the water generally work
through the successive layers. For TRANSPORTING THE CON
mixer to the mold, steel wheelbarrows, each holding about 2 cu
employed. A larger vehicle, holding about 6 cu ft, is made b
Concrete Machinery Company, Dunellen, N. J., and is found
in larger work. When the conditions will permit, concrete may
uted by means of CHUTES, but care must be exercised to secu
that will prevent the separation of the coarse aggregate from th
transporting through the chutes may be done either by grav
pressed air.

Pouring the Concrete. Ideal conditions would obtain i
PLACING CONCRETE could be CONTINUOUS. This is not gener
so it is important that the point at which work is stopped each
selected and predetermined that the strength of the construc
least. In smaller buildings, with floor-areas not exceeding abo
should be possible to so arrange the progress of the work that
construction may be placed in one day. In larger work it is
off a certain area to be completed within the time of concreti
Work should not leave off across important beams or girders,
rary stopping should be arranged for when the work is at the
or minor floor-beams. If any parts of floor-slabs are considere
tions for the strength of the beams or girders, such parts mu
at the same time and must be considered parts of such b
Joints in columns should be made perpendicular to the axes
and, as far as possible, at the lower side of girders. Columns sh
to set for at least two hours before girders are cast on them, in
for settlement and shrinkage.

Ramming the Concrete. As soon as the concrete has been

of Concrete Surfaces. The EXPOSED SURFACES of concrete sly treated in attempts to produce a satisfactory appearance. l provision is made, the marks of the lumber used in the forms in to show, and the lines of demarcation between successive ly defined. To eliminate these lines, grooves are sometimes d, by tacking on the sides of the molds triangular or trapezoidal ice sunk joints in the wall, and give it an appearance resembling The successive layers of concrete are in such cases stopped at at the junction of the two layers is hidden. In some cases rposely left rough and scratched like the scratch-coat in plaster- uccoed with a neat cement or a rich cement mortar. In this ere is always some danger that the stucco will flake off. The mes from the mold, is sometimes hammer-dressed, or rather pecial hammer. This hammer has an edge at right-angles to the edge is indented and made a series of points. A roughened uced which in time shows a uniform texture. Another method yed is to remove the forms as soon as the concrete is sufficiently the surface with a plasterer's float, using fine sand between wall-surface and plenty of water.

f Reinforced-Concrete Floors. If the FLOOR-SURFACES are d with a wooden flooring, a satisfactory finish may be obtained the surface, before the concrete has had time to set thoroughly, from 1 to 1½ in thick, and troweling to make it smooth and ish is attempted after the concrete has set, the new and the old ly not bond; and there is always danger of flaking off unless the considerable thickness.

and New Concrete. Various fluids and special cementitious een put on the market for the purpose of BONDING NEW AND OLD CES. Whether or not these materials have any special merits, ly accepted that a good rich cement mortar will form sufficient

and particularly of placing the reinforcement exactly as planne
sufficiently alert and active to see that the work of the contracto
properly; so that, for instance, work shall not have to be rebuilt
in the forms. The MATERIALS used in the construction, particula
should be tested as the work progresses. Cubes of the concrete
be made up each day and at the end of seven days should be
pression, and if necessary again at the age of twenty-eight day
will serve as a guide in the acceptance of the work, or in decidi
sity for a load-test of the finished structure. Under no circumst
should it replace or serve as an excuse to omit the testing of t
delivery or before acceptance. In addition to the details d
chapter, details which require the attention of the inspector on
others may be especially mentioned here:

(1) In JOINING NEW WORK with that which is already in, and
to set, the surface must be thoroughly cleaned and wet. In stop
is good practice where possible, to cast a groove in a surface tha
with another, so that when the work is afterward continued,
groove junction is effected.

(2) All FORMS or MOLDS must be carefully cleaned out just bef
is poured. The bottoms of the column-molds must be especia
this, as shavings, sawdust and even blocks of wood are liable t
unobserved. It is well to leave off a small piece of one side of t
at the bottom, for purposes of observation and cleaning, and to
before pouring the concrete.

(3) Great care should be excerised in POURING and RAMMING
molds, such as for columns, walls, etc., in order to get the m
filled at the bottom. In careless work it is not unusual to fin
very porous concrete, if not large pockets. This is particularl
when there is considerable reinforcing-steel in the construction.

(4) It should be remembered that concrete shrinks in setting.
at the tops of columns are sometimes found to be due to this c
are not always observable from the outside after the forms are
care should be exercised to guard against them. In pouring
molds should be filled to overflowing to the top of deep molds.

(5) The exact position of the reinforcing-steel in the concrete
importance that particular mention is again made of it here. I
struction the greatest care must be exercised, in the first place, t
forcement carefully placed, and then to avoid its being shifted
by the pouring and the ramming of the concrete.

(6) The REINFORCING-STEEL of those systems in which the a
tached stirrups is claimed, is often, for convenience in shippin
stirrups and laid flat or close to the main bar. It is intended
them on the job the stirrups shall be turned up to their proper
less carefully inspected, this is liable to be neglected.

(7) The use of a UNIT TYPE of construction (see page 926) pra

ibe in his specifications that such design shall be checked and
an engineer appointed by him. A fair load to be applied in a test
e-half times the working live load plus one-half the weight of the
The stresses in the construction are then equal to one and one-
e working stresses assumed in designing. Under these conditions
not be any evidences of distress, and the deflections should not
he span. The material used for the load-test should be so selected
hat, when uniformly distributed, as required, it will not arch and
mpressive strength of the beam or floor. Pig iron is a very good
ise. Bricks are more generally available, but must often be piled
get the required load, consuming much time and labor in making
hen bricks are used they should be set in vertical piles with spaces
between them, thus avoiding all arching of the load.

CHAPTER XXV

REINFORCED-CONCRETE FACTORY AND CONSTRUCTION*

By
EMILE G. PERROT
MEMBER OF AMERICAN SOCIETY OF CIVIL ENGINEE

General Principles. The problem involved in the proper d
forced-concrete factory or mill is a far more difficult one tha
from a superficial examination of the finished structure. This a
ings constructed wholly or in part of reinforced concrete, and is
that maximum economy and efficiency in production can on
when the building is thoroughly adapted to a given occupancy and
and even some architects, look upon the factory as a mere work
of four walls with floors and roof. To them it seems an easy m
the structure with reference to the lot or site and then supply it
elevators and kindred features. This, however, is not the case.
uses processes peculiar to itself. The ease with which these p
employed renders the profit-making more or less successful; he
sary to design the building to suit them. However, as the pu
explain what constitutes proper design, as applied to the rei
construction of a factory or mill-building, a typical case will serv
the principles involved. This chapter, therefore, deals with su
as would seem to meet the needs of the greatest number of pers

Walls, Floors and Roofs. Reinforced-concrete constructio
for walls and floors, or for floors and roofs only, in the latter c
for reinforced-concrete walls some masonry construction such as
It is not always advisable to use reinforced concrete for walls.
very frequently arise in which it is more suitable and economic
walls or piers.

Types of Floor-Construction. The floor-construction m
into two general types, the BEAM-AND-SLAB type and the GI
The beam-and-slab type may in turn be divided into varieties.
may consist of beams supported by columns, with slabs spanni
beam. This arrangement corresponds to simple mill-construction
the heavy timbers run across the building every 8 or 10 ft. T
on the wall at one end and on a post at the other, with 3 or 4-in
spanning from beam to beam. The earlier types of reinforced-
were patterned after this system. The next method was the
girders running from column to column, and the placing of the

eams are arranged along four sides of a square, a column being ch of the four corners. The simplest type of reinforced-concrete for factories is some form of the beam-and-slab type with walls and orced concrete. The GIRDERLESS type consists of a heavy flat slab columns without the use of beams or girders. The column-head form a large bearing-surface and the columns are spaced so as to bays as near as possible. A typical example is worked out at the hapter.

In general, as few columns as possible should be used to support ler that they may not interfere with the placing of machinery, and most economical use of the floor-space. From the standpoint of

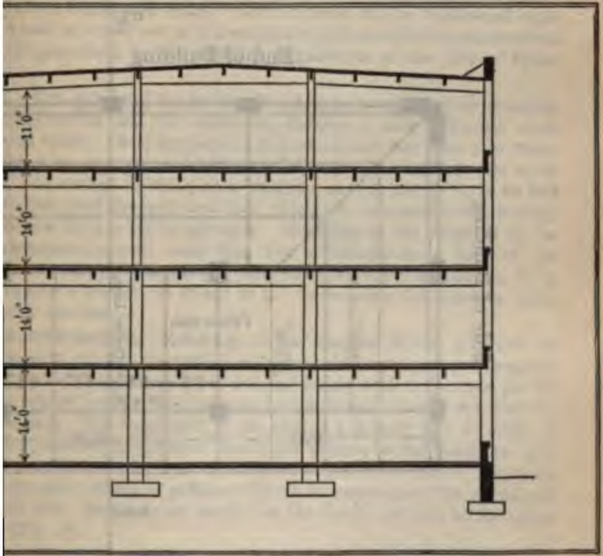


Fig. 1. Cross-section of Building

construction, however, the use of one column to not more than floor-space has been found to meet average requirements. This, s not include construction of a special class. Adopting this, then, rd, and bearing in mind the fact that the nearer a building comes re in plan the less is the total length of exterior wall required to en area, it can be assumed that a four-story building 75 ft wide s of columns, making three spans across the building, is a suitable purposes. (See Fig. 1.)

ting of a Building of this width, with story-heights of 14 ft, ill be ample for most purposes. There are always some parts of e for which a strong light is not absolutely essential and which d to aisles and to the storing of material in process of manufacture. art of the floor-space is generally used for this purpose, while the machinery is placed nearer the windows where the light is best and where the work is done. It is usually better, therefore, not to have a row of colum along the central axis of the building, unless it is definitely known that such arrangement will not interfere with the proper use of the floor-space. In a building 75 ft wide, two rows of columns, with spans of 25 ft crosswise of the structure, leave the central part of the floor-space free. Dividing 25 into 400 sq ft, the floor-space allowed for each column, gives 16 ft as the distan between columns, measuring lengthwise of the building.

Bays. The reason in this instance for making the bays rectangular instead of square is that there would be another row of columns if a square bay with a maximum of 20 ft in either direction were assumed. This would be likely t

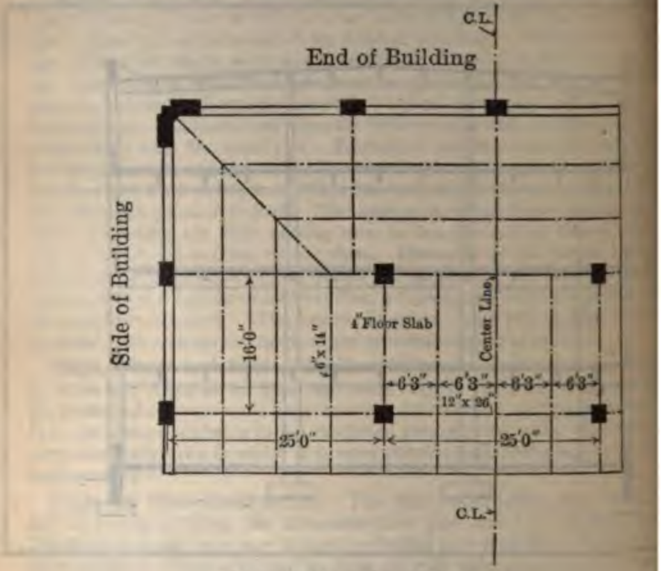


Fig. 2. Part Floor-plan of Building

interfere with the judicious placing of machinery and would result in a row columns along the central axis of the building. This is not considered g practice and should be avoided, except when there is to be only one row columns in the building.

Example of a Typical Bay The design of a typical bay of the size me tioned above, 25 by 16 ft, will now be considered. Referring to the illustrati (Figs. 1 and 2), it is seen that the windows occupy the major portion of the w area, the sill being set much lower than is usual in brick buildings. Thi done to avoid the necessity of the construction of an extra-high spandrel be as the lintel over the windows below performs the double function of support the floor and forming a curtain wall. The head of the window is carried to the under side of the floor-slab to simplify the construction of the bott

and at the same time permit the window to extend to the ceiling,
ducing the light at the highest possible point and allowing the rays
into the room. The first beam should be set as far back as possible
side wall and windows, so that the angle of the direct rays of light
arly horizontal as practicable. It will be found best to have the
run across the building, bearing on the walls and interior columns.
may be made as deep as economy of design suggests, as they run
the light-rays and do not interfere with the lighting-scheme. Again,
is relatively very economical. It also acts as a stiffener across the
ension of the building, thus increasing the resistance to vibration
oving machinery.

Floor-System. The various elements of the floor-system consist
irders, beams and slabs. Each of these will be considered sepa-
e load of 120 lb per sq ft is ample for light manufacturing purposes.
oad prescribed by the Building Regulations of the City of Phila-

. The spacing of the beams should be governed both by economy
construction and the maximum distance a slab will span while
load safely. It is impractical to make a slab less than 3 in thick.
ght, with concrete weighing 150 lb per cu ft, is 37½ lb per sq ft.
a 1-in cement finishing-coat, weighing 12½ lb per sq ft, to be laid
te, the total live and dead load which the slab must carry, if it is 3
o lb + 50 lb = 170 lb per sq ft. Referring to the diagram of the
einforced-concrete slabs (Fig. 18), calculated on a basis of the
ent equaling $Wl/10$, no curve is found in the 3-in diagram for a
o carry a load of 170 lb per sq ft. Some other slab must be used,
carry the load.

-Reinforcement. Referring to the diagram of the 4-in slab in
ollowing the 6-ft line until it intersects the horizontal line opposite
sq ft, it is found that a 4-in slab, reinforced with 0.195 sq in per lin
in square bars per foot, will carry slightly more than is required for
uestion. The total load, if the slab is 4 instead of 3 in thick, is
sq ft, and as the 187½-lb line is the nearest to this load, the 4-in
d as above, is adopted. The reinforcing-rods are placed 1 in from
f the slab and are of sufficient length to extend over two spans and
ach end; the joints are made over the beams and not in the space
n (Fig. 3).

s. The beams running from girder to girder are considered next
e span, center to center of girders, is 16 ft, and the distance apart
king an area of 100 sq ft carried by each beam. To the load per
f 182½ lb must be added the weight of the beam itself, which is
e 15 lb per sq ft of floor-area, making a total of 197½ lb per sq ft
by the beam. This multiplied by the area, 100, equals 19 750 lb.
moment caused by this load on the beam, based on the formula $M =$
for partially restrained beams is the one generally used, is 379 200
lab acts with the stem or beam to form a T beam and hence is as-
he compression-flange of the girder; and as the slab is 4 in thick, the
eam and the amount of reinforcement can readily be found by refer-
1, which is the diagram of the strength of T beams having a 4-in
am-depth in the diagram is the depth of the stem below the slab.
m opposite the center of the space between 350 000 and 400 000
nd side, the depth of beam that best suits the conditions can be
at the bottom *of the diagram is given the total area of steel to*

be used in the reinforcing-rods. As the depth of a beam from the economical use of material should be about one-twelfth the span, a or 16 in deep is found to comply with this rule. Below the space where the representing the 14-in depth of beam intersects the line representing the ing moment, it is seen that the area of steel necessary is 1.8 sq in.

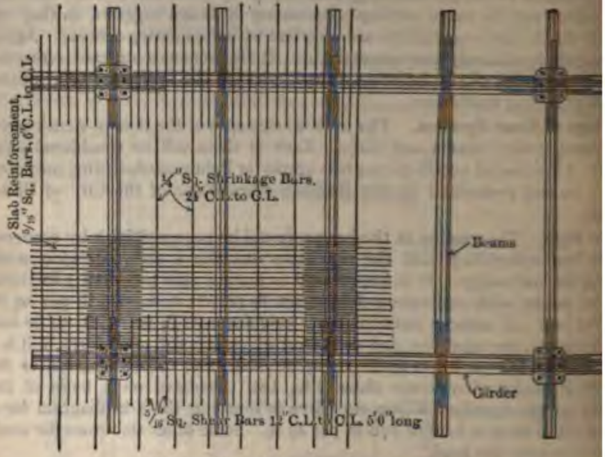

Slab Reinforcement, 5/16" Sq. Bars, 6" C.L. to C.L.
1/4" Sq. Shrinkage Bars, 24" C.L. to C.L.
Beams
Girder
5/16 Sq. Shear Bars 12" C.L. to C.L. 5'0" long

Fig. 3. Plan of One Bay, Showing Reinforcing

this over four bars, each bar should contain 0.45 sq in. The area of one square bar is 0.47 sq in, and hence a beam 14 in deep, reinforced with four square bars, is used. The width of the beam should be 6 in. A safe determine the width of the beam-stem is to allow 1½ in of concrete on the sides of the bars and arrange the bars in two rows, if the beams three or more bars. The distance in a horizontal direction, center to bars, should be 2½ times the diameter, but in any case there should be space between the bars horizontally, to permit the concrete to thoroughly them.

Arrangement of the Bars. Assuming the bars to be twisted, the center to center, of the two bars is 1⅞ in. Adding to this the diameter bars on their diagonal, which is about 1⅛ in, and 3 in for the sum is 6 in as the width of the beam required in this case (Fig. 6). It perfectly practicable to arrange the four bars in one row across the the beam; but the width would have to be 9¾ in, which is wider than requires. An additional objection to the latter arrangement is that it more concrete, thus adding to the dead weight of the construction. should be 2 in of concrete under the bottom of the rods for fireproofing.

Width of Beam. Of course, the width of the beam must be permit easy pouring of the concrete. Where wooden-box forms are used not good practice to make beams narrower than 6 in. If the beam is very say 36 in, 6 in would be too narrow a width in which to place the clean out the beam-forms. Practical considerations very frequently the width of beams.

. There should be in each beam and girder a sufficient number of ade of at least 5/16-in round bars, bent U-shaped, run under the ds and extended up into the slab with an angle-bend 6 in long. If r girder is short and excessively deep, 3/8-in round or heavier stirrups used. The function of stirrups is to unite mechanically the slab to so that perfect T-beam action will result, and also to assist in the to DIAGONAL TENSION or SHEAR as it is commonly called. The num- ups in a beam should be approximately one for each foot of the span, center, but the spacing should be as stated below. Thus, a 16-ft ld have not less than sixteen stirrups, that is, eight on each side of line.

Spacing for Distributed Loads. For beams with distributed stirrups are to be spaced so that the minimum distance between them in in ordinary the maximum ot more than 36 middle of the ch half of the ld be divided parts. The earest the sup- ld contain ap- y one-half the stirrups allotted f the beam, or the total num- middle division ntain one-sixth umber, and the xt to the cen- ne-twelfth the er, as shown in the distribution ork out evenly g which comes to this should

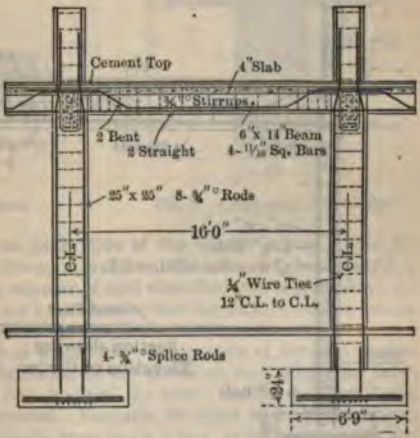


Fig. 4. Section Showing Elevation of Beam

Spacing for Concentrated Loads. When there are concentrated stirrups should be designed to suit the loading, but in any case, nce equal to about one-fifth the span from each end, the stirrups spaced at least from 4 to 6 in on centers. A good rule to follow is e side of safety and to put in plenty of stirrups, if the determination t number is in doubt, as there should be a sufficient number of them at part of the diagonal tension not safely resisted by the concrete.

rangement of the Bars in the Beam is shown in Fig. 4. The bars are bent upwards near the supports to resist the negative bending hich causes tension at the top of the beam near the supports. These l extend into the next span at least 30 in to form a tie. As rein- rete is of a monolithic character, it is necessary to introduce metal ver the concrete is subjected to tensile stresses. While it is not o provide as much steel at the top of the beam over the supports as for restrained beams gives, if *50% of the area in the beam is carried* to the top and over the supports, as shown in the illustration, the perfectly safe when calculated on a basis of M equaling $Wl/10$. In beams must be calculated on the basis of $M = Wl/8$. Then it is sary to have about one-fourth the number of the bars bent up near These bars, however, should extend at least 30 in beyond the girder or column to tie the building together.

For Simple Beams with Uniformly Distributed Loads, 60% of the span should be straight and the truss-rods should bend points so determined.

For Beams or Girders with Concentrated Loads, all bars are as far as the concentrated loads extend. Beyond these loads, tow ports, one-half the number of bars may be bent up as above.

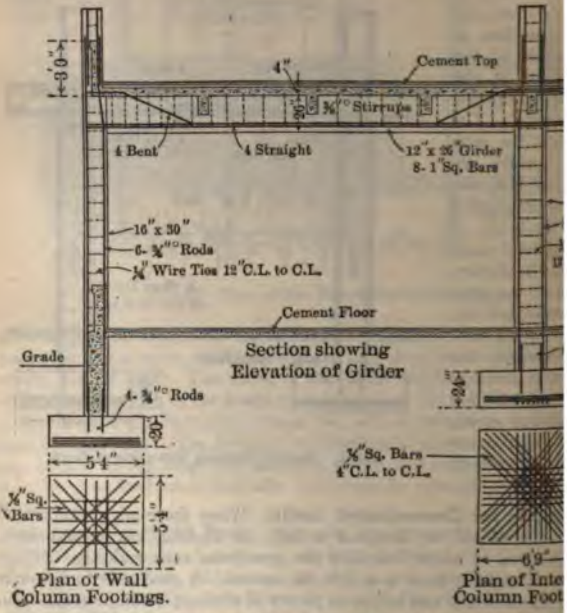


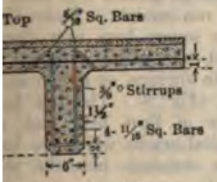
Fig. 5. Elevation of Girder and Plans of Column-Footings

The Girders. The girders running across the building are calcu basis of carrying their own weight as a uniformly distributed lo centrated loads at the points where the beams frame into them. the illustrations, Figs. 2 and 5, it will be noticed that there are thr each side supported by the girder, the fourth beam being carried by Each concentrated load equals the total load on the beams, or 19 weight of the girder can be assumed as 20 lb per sq ft of area carri = 8 000 lb. This acts as a distributed load. One-half the span of

. The bending moment at the middle of the girder from the con-
d distributed loads is

$$9\,625 \times 150) - (19\,750 \times 75) + \frac{8\,000 \times 300}{8} = 3\,262\,500 \text{ in-lb}$$

g-Bars and Width of Girder. Referring again to Fig. 21, in the space opposite 3 300 000 and 3 250 000, the line of a 26-in deep n to intersect the vertical line representing 8 sq in of steel. Hence uare bars arranged in two horizontal rows are used. The width of e 12 in in order to have the proper distance between the bars and ime have 1½ in of concrete fireproofing on the sides (Fig. 7).



Cross-section of Beam

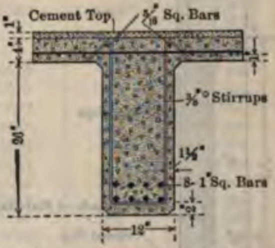


Fig. 7. Cross-section of Girder

th of the Slab on each Side of the Girder is found by multi- ea of the steel by the number on the line of the 26-in beam, which is nstant is used for any area of steel when the beam is 26 in deep; the the other beams are to be likewise used for their respective beams. f this girder, the width of the T beam is 8.7 × 8 = 69.6 in, or 34.8 ide of the middle of the girder. The portion of the slab used at of the girder or beam should not exceed on each side of the beam e slab-thickness, nor one-third the span. In the case now being he limit is not exceeded. Similarly the width of the slab acting as sion-flange of the 14-in beam is 2¼ × 12 = 27 in, twelve being the 14-in beams.

els. The next member to design is the lintel, or spandrel beam over Figs. 2, 5 and 8). This should be, for practical considerations, 6 in e bottom of the lintel is flush with the bottom of the slab, the slab- n into the lintel over the top of the lintel-rods. In addition to the e lintel there should be bars of the same size as the stirrup-bars, 12 in apart and bent at right-angles, one leg extending up 12 in l and the other 18 in out into the slab; or else the slab-bars should extending into the lintel 12 in. These make a perfect tie between lintel. The bottom of the lintel should be made with a rebate to ead of the window-frame. The load carried by the lintel is the e slab, the weight of the window and the dead weight of the lintel. n the floor-slab is 13½ ft (the clear span of the lintel) × 3 ft = 40½ lb, the load per square foot on the floor-slab, or a total load from of 7 371 lb. The total height of the lintel to the window-sill is 3 ft. thick this makes the weight per lin ft 75 × 3 = 225 lb, the total lintel being 225 × 13½ = 3 038 lb. For the window 10 lb per sq ft The area being 13½ × 11 ft, *the height of the window, or in even*

figures 149 sq ft, the weight is 149 × 10 lb = 1 490 lb. The total
lintel, then, is 7 371 + 3 038 + 1 490 = 11 899 lb.

The Lintels Figured as Rectangular Beams. By referring to the
Explanation of Diagrams and Formulas, page 992, for the strength of
beams, it is seen that when reinforced with 0.5% of steel the safe load
the beam is $W = wl = 48\frac{bd^2}{l}$. Therefore, a 6 by 30-in beam will carry
= 15 552 lb. The depth 27 is used, as it is taken to the center
the steel. This is more than the load upon the lintel and hence
safe. A reinforcement of 0.5% equals 0.005 of 162 sq in, the area of th
or 0.81 sq in; and if two bar
each must be of 0.4-sq-in sect
Two 5/8-in square bars, each
area of 0.39 sq in, will be us
should be located 2 in from th
and run straight. There sho
3/8-in square bars near the
lintel, running the full length, a
5/16-in stirrups, as shown in th
tion (Fig. 8). The top bars
place of bent bars and al
vertical cracks which are liab
from shrinkage near the mid
lintel.

The Columns. Having
the design of the floor-system,
sions of the wall piers, inter
and footings are next dete
schedule of the loads on
columns will now be made.

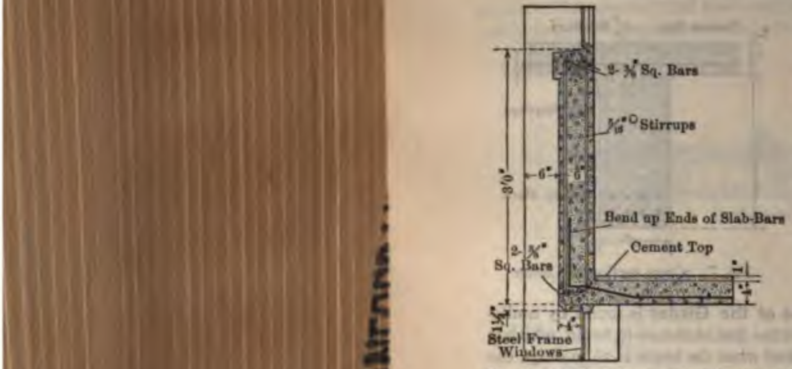


Fig. 8. Vertical Section, Showing Lintel

The Load from the Roof. Assuming a live roof load of 30
and 10 lb additional for accidental load from overhead shafting, th
load is 40 lb per sq ft. The weight of the slab, if 3 in thick, which
as is usually required, is 37½ lb per sq ft. The beams and girders we
30 lb per sq ft (12 plus 18), making a total dead load of 70 lb, i
covering. Adding the live load of 40 lb to this gives 110 lb per sq ft
dead and live load.

The Load on the Fourth-Story Column, then, is 400 time
44 000 lb, not counting the weight of the column itself, as for prac
no column should be made less than 10 by 10 in in cross-section
therefore, 500 lb per sq in unit stress on the concrete for columns,
unit stress allowed by the Philadelphia Building Bureau in reinfor
columns with vertical reinforcement, the carrying capacity of a
column is 100 times 500, or 50 000 lb, which is in excess of the load t
(See Table I.)

The Load on the Third-Story Column is the load from th
of 44 000 lb plus the load of one bay of the fourth floor, which is
= 86 800 lb, being the total dead and live load; or 86 800 + 44 000 =
which must be added the weight of the column, which is assumed to
lin ft. As it is about 11 ft long in the clear, the weight of the colum
which, added to 130 800 lb, equals 134 100 lb. The area of the cro
a 16 by 16-in column is 256 sq in, which, at 500 lb per sq in, gives

ing capacity. While this is 6 100 lb less than the load to be carried, ½% of the required strength. It is customary to make a reducoad to be carried on the columns in proportion to the amount of ried, the reduction being greater as the floor-area increases. Usuluction of the LIVE LOAD per floor, with a maximum not exceeding ottom columns for high buildings, is considered good practice.

Table I. Strength of Reinforced-Concrete Columns

th vertical bars. Safe working stress on concrete 500 lb per sq in, the rods being neglected in figuring the columns

Area	Total safe loads in lb	Size	Area	Total safe loads in lb
64	32 000	18×18	324	162 000
81	40 500	19×19	361	180 500
100	50 000	20×20	400	200 000
121	60 500	21×21	441	220 500
144	72 000	22×22	484	242 000
169	84 500	23×23	529	264 500
196	98 000	24×24	576	288 000
225	112 500	25×25	625	312 500
256	128 000	26×26	676	338 000
289	144 500	27×27	729	364 500

on the Second-Story Column is 134 100 lb plus the load from r and the weight of the columns, all of which is assumed as being fourth-floor load and weight of column, or 90 100 lb, making the lb. A 21 by 21-in column will carry 441 times 500 lb per sq in, .

on the First-Story Column is 224 200 lb plus the second-floor o lb and the weight of the column, which, at 600 lb per lin ft, is total of 317 600 lb. A 25 by 25-in column will carry 625 times q in, or 312 500 lb, which is almost the required strength. The lule then becomes

For the first story 25 × 25 in in cross-section.
For the second story 21 × 21 in in cross-section.
For the third story 16 × 16 in in cross-section.
For the fourth story 10 × 10 in in cross-section.

forcement in the Columns should consist of eight ¾-in round wo lower and four in the two upper stories, with ties of ¼-in round in, as shown in Fig. 9. It is the custom to use the same unit forced-concrete columns up to 15 diameters, and not to use columns exceeds 15 diameters.

Piers. The schedule of all the wall piers is made by the for the interior columns. The details of the calculations are not re, results only being given. The size of the wall piers is detere architectural effect desired and by practical considerations. in as the smallest face-dimension of the piers, this size should the full height of the building (Fig. 10). The reveal of the piers rels is 6 in, and the spandrels should line up flush with the inside if by so doing *they are not made extremely thick.* Reinforced-

concrete spandrels may be 6 in th
make them thinner than this, on a
It is to be noticed, also, that the
wall pier to another. They should
vertical loads coming upon them,

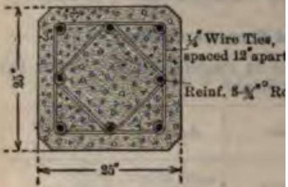


Fig. 9. Interior Column

tion in the direction of the length
resist the vibration in the directi
quently the main girders are run
the shortest distance, while the
this will make the construction
the windows in the side walls due
to permit the beams to be carried
as the light from the windows in
are run up to the under side of th

The wall-pier schedule, figured

For the first sto
For the second
For the third st
For the fourth

It will be noticed that the pier
dimensions. This is due to prac
the spandrel being 6 in and the
must be 12 in in order to be flush

Spread Foundations. The u
a building results in economical co
base or footing more than is custo
some kind. In order to give suffic
tions for the building under discus
of construction met with in pract
following pages. The simplest fo
shown in Fig. 5 and consists in
footings as CANTILEVER BEAMS.
signed as explained in the followin

The Load on the Footing.
317 000 lb and the safe bearing val
a spread footing of 317 000 lb div
square which comes the nearest to

The Design of the Footing.
square foot of footing sustains an
portion is treated as a CANTILEVER.

roper causes no bending, and this load is neglected in finding
t. The rods should be run as shown in Fig. 5, some diago-
ight-angles to the sides, the first layer located 3 in from the
ng. The size of the rods on the diagonal is now to be de-
thers are to be made the same size. The longest length of
al cantilever is 4 ft, measured from the center of the column
f the 1-ft-wide strip with the side of the square. The bend-
strip is equal to the load on an area, outside of the column,
ide, or (3 × 7 000 = 21 000 lb) × 30 in = 630 000 in-lb, 30 in
rom the axis of the column to the center of gravity of the

ting to be 24 in thick over-all, the CENTER OF ACTION of the
in up from the bottom, making an EFFECTIVE DEPTH of 19 in.
the steel is nine-tenths of the depth when the stress in the
er sq in, the resisting moment of the steel per square inch
6 000 lb is 16 000 × 0.9 = 14 400 lb. As the bending
n-lb, the number of square inches of steel necessary per foot
$\frac{00}{\times 19} = 2.34$ sq in. This formula is for rectangular beams
oment is given. (See Formula (1), page 992.) Spacing the
requires three rods per foot, each requiring a cross-section
As a ⅞-in square bar has a section-area of 0.76 sq in, this
he bars in the layers at right-angles to the side are made the
d as above, so as to avoid complications in the construction
would be possible to space these farther apart, but this re-
ary. (See Fig. 5.)

a column is such as to require a footing more than 2 ft thick,
lope the top of the footing, thus saving in the quantity of
provide a concrete PLINTH or BLOCK at the bottom of the
e footing so as to reduce the projection of the footing and
ore economical design. If steel column-cores or hooped
al reinforcements are used, a metal base-plate is necessary
g of sufficient size to limit the direct stress on the footing to

s for the Outside Walls may be designed in either of
CONTINUOUS FOOTINGS such as are usual in ordinary construc-
as ISOLATED PIERS under the wall columns. In the first case
einforce the footings and foundation-walls, as these act as
loaded at each column, and must be made strong enough to
from the columns uniformly over the entire length of the
dation-walls and footings can be treated as INVERTED CON-
g. 11), the upward reaction of the earth being considered a
ed load on the beams, and the wall piers being considered as
the beams, with the load on each pier as equal to the load on
. 12 shows the arrangement of the reinforcing-rods. Their
s explained in the following paragraph.

r running foot of the foundation is equal to the load from a
distance apart of the piers, omitting the weight of the spandrel
windows, this load per running foot = 191 140 lb, the load
ft = 11 946 lb. As great refinements in calculations are not
work of this kind, because of the advisability of large factors
rt of the building and the small reduction in cost due to any
strength of this continuous beam is calculated by the formula

$M = Wl/8$, assuming l to be the clear distance between the pie 13 ft 6 in (Fig. 12). Therefore, $W = 13\frac{1}{2} \times 11\,946 = 161\,271$ ing moment $M = (161\,271 \times 162)/8 = 3\,265\,737$ in lb. As th is determined by the thickness of the wall and its depth, all tha

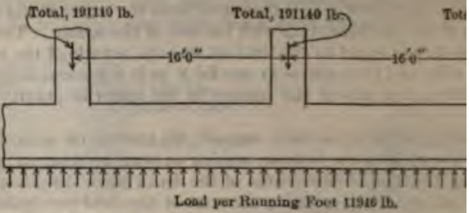


Fig. 11. Foundation-wall an Inverted Continuous B

find the AREA OF THE STEEL by referring to Formula (1), pag $A = \dfrac{M}{14\,400\,d}$, or $A = \dfrac{3\,265\,737}{14\,400 \times 52} = 4.3$ sq in, distributed in bars with a total area of cross-section of 4.48 in. These are i running straight and four bent as shown in Fig. 12. The top l

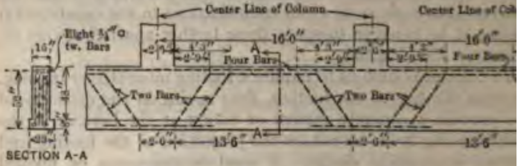


Fig. 12. Arrangement of Rods in Foundation-wa

from the top of the concrete. The footing is made wider tha the load on the soil within the safe limit of 7 000 lb per sq determined as follows. As the column-spacing is 16 ft, center to = 112 000 lb, the load the foundation 1 ft wide and 16 ft long to carry 212 120 lb (the load from the pier, plus 20 980 lb, spandrel and footing), 212 120 is divided by 112 000, giving 1 of the footing, or 1 ft 11 in, nearly.

Isolated Piers. In the second case, a SPREAD FOOTING each wall column in the same manner as under the interior colu for the lighter load. The foundation or spandrel wall is not in the first case, as it carries no load except its own weight and t above it. (See Fig. 5.) WHERE THE SOIL IS BAD and of low the PIER-METHOD is found to make an economical foundation

e of the footings. In order to secure uniform soil-pressure it is
to combine an interior with an exterior column-footing so as to
ad uniformly from the two columns to the soil below. Some-
sary to combine the footings of more than two columns. Fig. 13
s of an actual construction and may be regarded as typical. The
columns in this case are almost identical, one being 700 000 lb

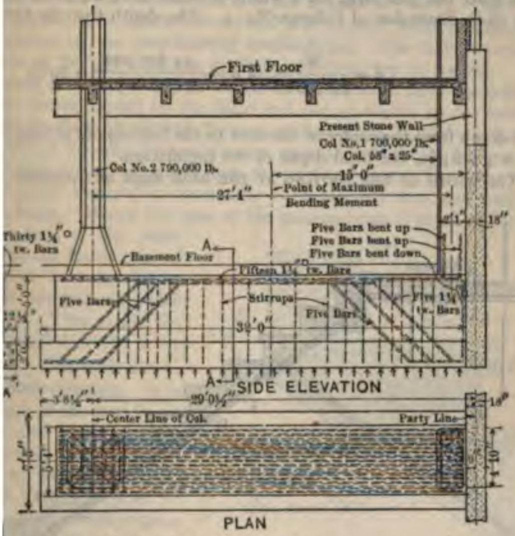


Fig. 13. Combined Column-footing

o 000 lb, so that the shape of the combined footing in plan can
, as the center of gravity of the two loads is practically at the
an. When one column is more heavily loaded than the other,
avity of the loads is no longer at the middle of the span, but
n; hence it is necessary to make the combined footing TRAPE-
o that the center of gravity of the trapezoid will coincide with
of the resultant of the loads from the columns.

calculations for the design of this footing are the actual ones
as a good example of the necessity of assuming certain sizes at
he final calculations may change. The WIDTH OF THE FOUNDA-
mined by the LOAD-LIMIT on the soil, which in this case is not to
er sq ft, and the size of the column-base being known, we may
mine the BENDING MOMENT in the footing. We assume an area
4 sq ft, giving a soil-pressure of 1 490 000 lb ÷ 224 sq ft = 6 650
650 × 7 = 46 550 lb per running foot. The point of maximum
is where the vertical shear is zero and is determined by the
/46 550 = 15 ft. Also, 15 ft − 1.05 ft = 13.95 ft. Hence

700 000 × 13.95) = 9 765 000 ft lb] − [(46 550 × 15 × 7½)
236 875 ft lb] = (4 528 125 × 12) = 54 337 500 in lb

The 1.05 ft is one-half the column
We may determine the DEPTH O
sectional area of the reinforcing-steel
the depth. For practical considerat
should not be used; hence by trial w
section-area of 46.8 sq in, placed in
will space out just right for a width
the 58-in dimension of Column No.

$$d = \frac{M}{14\,400\,A}, \text{ or }$$

the depth from the center of the stee
80 + 4 = 84 in, the total depth of th
The WIDTH OF THE FOOTING AT TH

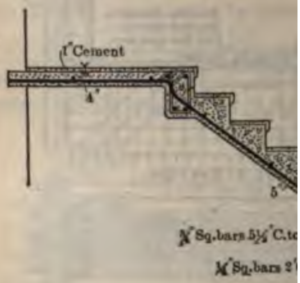


Fig. 14. Section through I

stress on the concrete in compression
As the total horizontal compression i
order to satisfy the requirements for
lb per sq in × 46.8 sq in = 748 800 lb.
the depth of the AREA OF THE CONC
= 24.8 in. The width is found by
the resistance of the concrete per inch
the width of the concrete at the bott
unit stress on the area of the concre

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o the corner formed by the tread and rise, as shown by the letter *D*

The landings are figured the same as floor-slabs. Their supporting calculated to carry the load coming upon them from the landing and pper stair-run, which starts from the landing-beam. The lower stair-g up under the landing-beam, acts as an inclined strut and supports this beam. Hence the span of the landing-beam is equal to the dis-n the wall of the stair-haft to the inside edge of un from below, and is a than one-half the width r-shaft. This makes the reinforced-concrete stairs mical. (See page 905.)

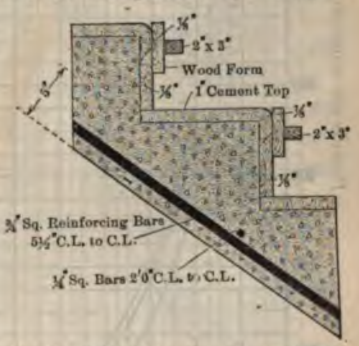
⅜"
2"x 3"
Wood Form
⅜"
1" Cement Top
⅜"
2"x 3"
⅜"
5"
⅜" Sq. Reinforcing Bars 5½" C.L. to C.L.
¼" Sq. Bars 2'0" C.L. to C.L.

Fig. 15. Detail of Reinforced-concrete Steps

e in Stair-Design. It is at each of the runs is 4 ft that the maximum live can come upon the stairs is one person, weighing each 2 ft of step, or 75 ft of step. With steps 4 e live load is 300 lb per ten steps, 10 times 300 per run for the live load. load is approximately 400 or 4 000 lb for the run. This makes a total load on the inclined beam . The span in calculating inclined beams is taken at the horizontal etween supports; hence in our example the span is 8 ft 9 in. The bending moment, therefore, is $\frac{7\,000 \times 105}{10} = 73\,500$ in-lb, figuring partially restrained. Assuming the thickness of the slab to be 5 in, the epth is 4 in, and the area of steel per foot of width for this depth and oment as above is $\frac{73\,500}{4 \times 14\,400} = 1.3$ sq in, approximately. If ¾-in

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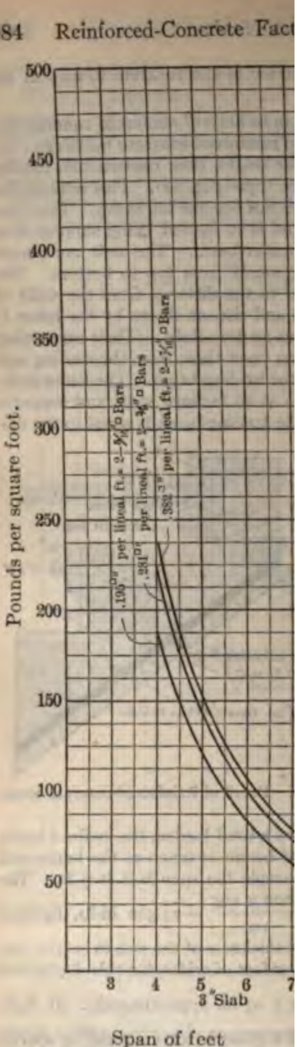
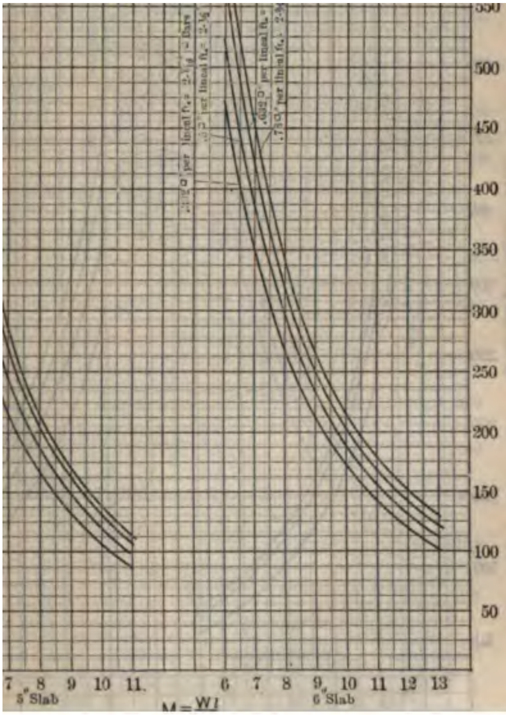


Fig. 16. Diagram for Str

when their forms are removed. T
as the concrete has set sufficiently t
or tread can be incorporated into
the concrete has set hard is very lik
good form of step is shown in the


.312□" per lineal ft.= 2-½ □ Bars
.5□" per lineal ft.= 2-½
.632□" per lineal ft.=
.75□" per lineal ft.= 2-¾
550
500
450
400
350
300
250
200
150
100
50
7
8
9
10
11
5" Slab
6
7
8
9
10
11
12
13
6" Slab
M = WI

These unit stresses give a factor
of the materials and have been fo
sistent with safety and economica

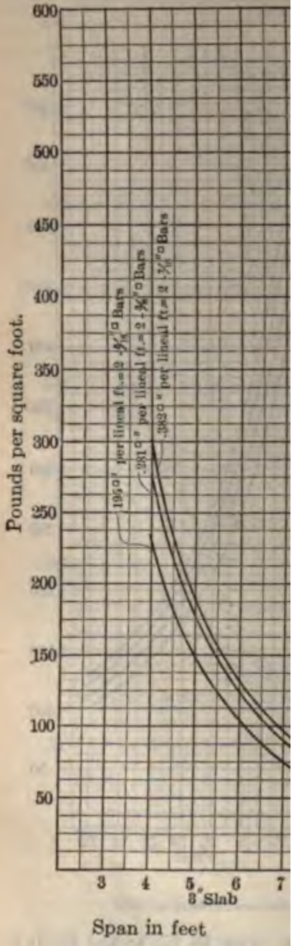
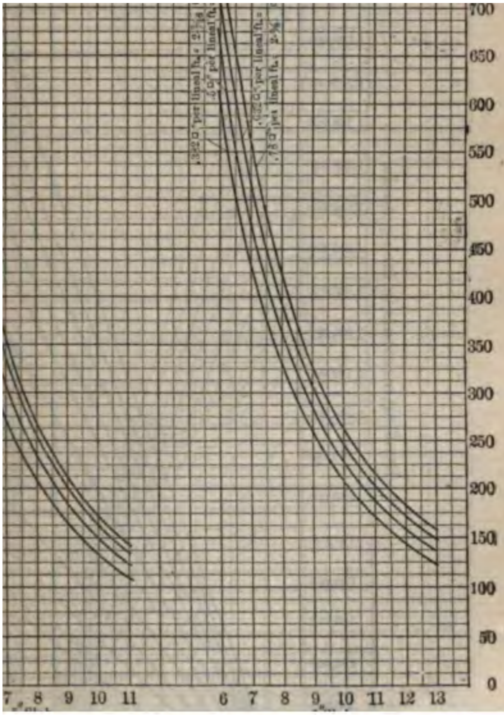
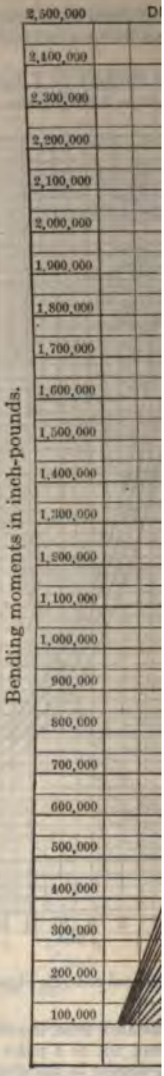


Fig. 18. Diagram for St

mixture and the aggregate a good
cities usually specify the allowable u
concrete structures, and when they



700
650
600
550
500
450
400
350
300
250
200
150
100
50
0
7
8
9
10
11
6
7
8
9
10
11
12
13
.352 □" per lineal ft.
.5 □" per lineal ft.
.635 □" per lineal ft.=
.78 □" per lineal ft.



Bending moments in inch-pounds.
2,500,000
2,400,000
2,300,000
2,200,000
2,100,000
2,000,000
1,900,000
1,800,000
1,700,000
1,600,000
1,500,000
1,400,000
1,300,000
1,200,000
1,100,000
1,000,000
900,000
800,000
700,000
600,000
500,000
400,000
300,000
200,000
100,000
D

Fig. 20. Di

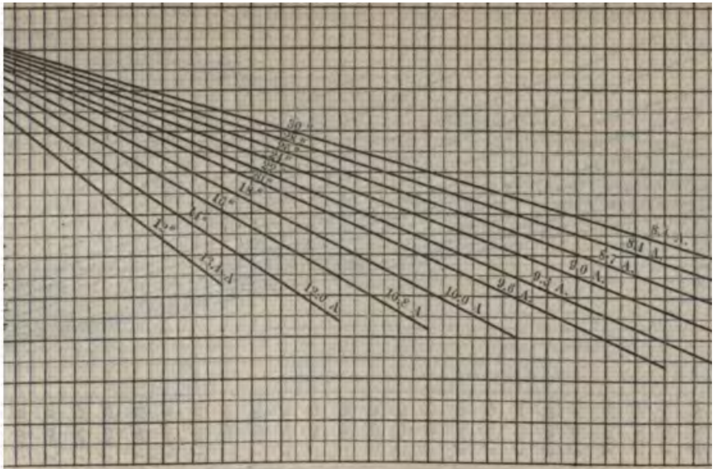

30"
28"
26"
24"
22"
20"
18"
16"
14"
12"
13.4 A
12.0 A
10.8 A
10.0 A
9.6 A.
9.3 A.
9.0 A.
8.7 A.
8.4 A.
8.1 A.

DIAGRAM O

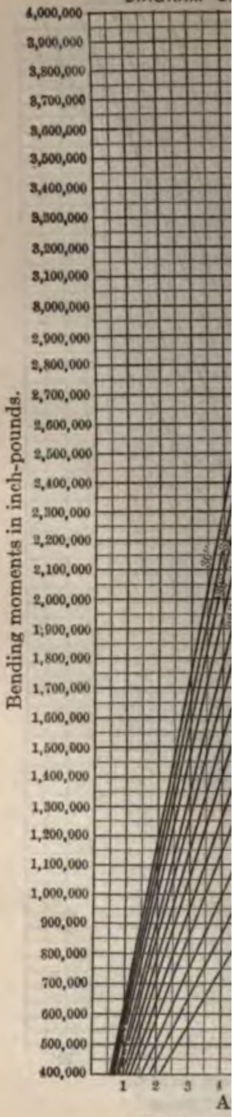
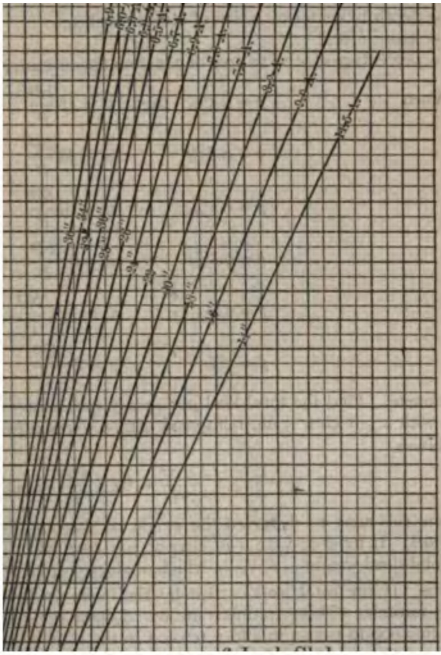


Fig. 22. Di



The following formulas are for the strength of rectangular beams based on various percentages of steel, the beams being considered to be supported at the ends. They are calculated in accordance with the Phila requirements, and can be used in investigating the strength of rect beams and slabs without obtaining the bending moment. They are venient in checking up a design already made, or in establishing the ar steel reinforcement when the size of the concrete beam or slab is fixed, by the example given.

w = load in pounds per running foot;
b = breadth of beam in inches;
d = depth to center of action of steel in inches;
l = span in feet;
p = percentage of steel to area of concrete above center of steel to to

When $p = 0.5\%$ then $w = 48 \frac{bd^2}{l^2}$

$p = 0.6\%$ $w = 56 \frac{bd^2}{l^2}$

$p = 0.7\%$ $w = 59 \frac{bd^2}{l^2}$

$p = 0.8\%$ $w = 62 \frac{bd^2}{l^2}$

$p = 0.9\%$ $w = 64.5 \frac{bd^2}{l^2}$

$p = 1\%$ $w = 67 \frac{bd^2}{l^2}$

Example. Find the total load per square foot that can be carrie slab, with a 5-ft clear span, reinforced with 0.8% of steel per runnin

Solution.

$$w = 62 \times \frac{12 \times 3^2}{5^2} = 62 \times \frac{108}{25} = 266.6 \text{ lb}$$

From this must be deducted the weight of slab and floor-finish to live load. If finished with 1-in cement top coat laid directly on t the total dead weight is 62½ lb, which, deducted from 266.6 lb, leav

Note. If the total load carried by the beam is desired, use l i in the formula. These formulas are based upon the stress in the c exceeding 600 lb per sq in and a tension in the steel of 16 000 lb per a ratio of the moduli of elasticity of the concrete and steel equal to

Formula for the Resisting Moment of Rectangular or T Be is Formula (6), page 936, Chapter XXIV, only in a different form, used when the percentage of steel is not greater than 0.58 of 1%.

M = the maximum bending moment in inch-pounds;
d = the depth from the top of the beam to the center of action of inches;
A = the area of the sum of the cross-sections of the steel bars in sq

$$M = A \times 16\,000 \times 0.9\,d \text{ or } A = \frac{M}{14\,400\,d}$$

or $$d = \frac{M}{14\,400\,A} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; .$$

e have been using, namely, 600 and 16 000 lb per sq in for the 12 for the ratio. This value of K, from Table V, page 930, s 83.4 and $M = 83.4\ bd^2$. Transposing, we have

$$\frac{}{d^2}, \text{ or } b = \frac{217\ 728}{83.4 \times (13\tfrac{1}{2})^2} = \frac{217\ 728}{83.4 \times 182.2} = 14.3 \text{ in}$$

efore will be 14½ in × 16 in in cross-section, reinforced with two placed so that there will be 2½ in from the bottom of the beam As the width of this beam is excessive for the number of rods omical. It would be better to design the beam with a T section h to 6 in for the stem and making the top flange 14½ in wide 4.18 in thick. The ratio of the distance of the neutral surface the beam to the effective depth of the beam, for the values we s 0.31 (see Table V, page 930, Chapter XXIV), and in order to ncrete in compression at the top of the beam to balance the ne steel, the head or flange of the T must extend from the top neutral surface.

oors.* In order to familiarize the student with the design of s, an example is worked out, in which the area of a panel or bay 400 sq ft, the same as that of a typical bay in the factory-building dered in this chapter. The column-spacing is made the same s, so that the panels are square, with a length of side of 20 ft. ng the various methods of computing the strength of flat, replates, we will use one under consideration by the Bureau of on of Philadelphia.† This is a conservative method. It has orked out in all its details and applications and gives results afety and economical design. The following paragraphs set and equations of this method as published by the Philadelphia ls it the DROP-CONSTRUCTION.

h, center to center of columns, of the longest of straight bands es.

nce or width, edge to edge, between capital-heads in inches.

If the drop-construction is not used, $d = d_1$.

Sufficient depth of slab is to be provided for shearing-stresses bending-stresses.

Width of capital-head = not less than $\frac{2}{10}L$.

Width of drop = $\frac{35}{100}L$.

Width of bands = $\frac{5}{10}L$.

x = the area of section of steel over the capital-head.

x_1 = the area of section of steel in center of bay.

$-M$ = the bending moment at the edge of the capital-head.

$+M$ = the bending moment at the center of the span.

The load carried by the straight band = $\frac{\text{total bay} - \text{capital-h}}{2}$

$$-M = \frac{\text{total bay} - \text{capital-head}}{2} \times \frac{wL_1}{12}$$

$$+M = \frac{\text{total bay} - \text{capital-head}}{2} \times \frac{wL_1}{24}$$

Width of concrete to resist compression at edge of capital- drop.

Width of concrete to resist compression when negative m = width of band + 3 T, in which T = the thickness of slab.

Width of concrete to resist compression at middle of span = + 6 T.

$$x = -\frac{M}{d \times 16\,000}$$

$$\left.\begin{array}{l}\text{Place } 66\% \text{ of } x \text{ in straight bands}\\ \text{Place } 43\% \text{ of } x \text{ in diagonal bands}\end{array}\right\}\text{over capital-head.}$$

$$x_1 = +\frac{M}{d_1 \times 16\,000}$$

$$\left.\begin{array}{l}\text{Place } 66\% \text{ of } x_1 \text{ in straight bands}\\ \text{Place } 43\% \text{ of } x_1 \text{ in diagonal bands}\end{array}\right\}\text{at middle of span.}$$

The drop equals the abacus outside of the capital-head, or the in of the concrete to obtain the necessary compression in the concr generally necessary when the live load of the floor is light, say and the span is not excessive. To determine d and d_1 deduc thickness of the slab 1 in to the center of the steel when the rod in diameter; if over ⅝ in deduct 1½ in; and multiply the res result will be the distance from the center of the steel to the ce

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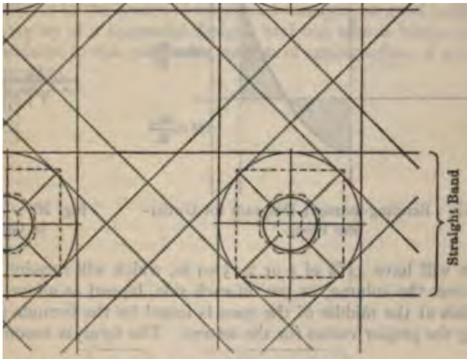

24. Arrangement of Bands in Girderless Floor

the bay = $20^2 = 400$ sq ft.

pital-head = $4^2 = 16$ sq ft. Then, by the formula, the load

ght bands = $\frac{400 - 16}{2} \times 233 = 44\,736$ lb

$$\frac{44\,736 \times L_1}{12} = \frac{44\,736 \times 16 \times 12}{12} = 715\,776 \text{ in-lb}$$

$$\frac{44\,736 \times L_1}{24} = \frac{44\,736 \times 16 \times 12}{24} = 357\,888 \text{ in-lb}$$

ent diagram is shown in Fig. 25.

xt to find the thickness of the concrete at the drop. The

The straight band will have 66% of 5 or

a cross-sectional area of 0.11 sq in. There

the capital-head in the straight band. As
lap the column-head, extending into the ne
each straight band over the column will h

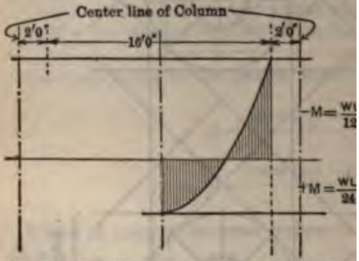

Fig. 25. Bending-moment Diagram for Girde
less Floor

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bands will have 43% of 5 or 2.15 sq in, w
rods over the column, or ten on each side
the slab at the middle of the span is foun
tuting the proper values for the letters. T

$$h = \sqrt{\frac{2 \times 357\,888}{0.27 \times 138 \times 600}} = \sqrt{}$$

The total depth is $5.6 + 1$ in = 6.6 in, o
The width of the band = 10 ft + (3 × 6
For the steel at the center of the span

$$x = + \frac{M}{d_1 \times 16\,000} \text{ in which } d$$

$$x = \frac{357\,888}{6.3 \times 16\,000} = 3.5 \text{ sq in}$$

The straight bands will have 66% of 3.

quire $\frac{2.31}{0.11}$ = twenty-one ⅜-in round bars

span than for the band set over the colum

In practice the rods are made the full
column, plus the width of the drop, or in th
for the fifteen rods. Six additional rods,
the edge of one drop to the edge of the nex
make the twenty-one required for the mid
will have in the center 43% of 3.5 sq in =
round rods or four more than one set of roc
ever, are to be added at the middle of th
are bent up over the column-head so as to
of the negative bending moment, the bar
of the slab the full width of the drop. It i
extending down into the column and out

wide experience in the design of reinforced concrete, as good judgment the making up of a successful design; and one who is inexperienced nsult a specialist in this particular system of construction, if a design t into execution.

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1. Definitions

Use of Trusses. Whenever the distance between the side
ing exceeds about thirty feet, and there are no intermediate wa
is usually necessary to support the roof on trusses. The ceiling
assembly-halls, etc., also, require trusses for their support. In
roof and a ceiling are carried by the same trusses.

A Truss is a framework, composed of straight, or sometim
bers or pieces so arranged that the structure as a whole acts a
a triangle is the only figure which cannot be changed in shape
the length of one or more of its sides, it follows that the pieces
must be arranged so as to form triangles. The members of a
subjected to longitudinal stresses only, either compressive or
members and members which act as beams supporting loads
additional bending stresses. Each member of a truss is either

A Tie is a member which has developed in it a longitudinal

A Strut is a member which has developed in it a longitud
stress. When vertical, struts are sometimes called POSTS or co

The Top Chord of a truss is composed of the upper outsi
some forms of roof-trusses top chords are called RAFTERS (Fig.

The Bottom Chord of a truss is composed of the lower
(Fig. 2). In roof-trusses the bottom chord is commonly calle

The Web-Members are those connecting the CHORDS (Fig.

A Joint is the point of intersection of two or more members o

A Panel is the distance between two adjacent JOINTS in eit
lower chords (Fig. 2).

Purlins. Whenever possible all roof-loads and ceiling-loads
ferred to trusses at the joints. This usually requires beams sp
between trusses at corresponding joints. These beams, when
roof, are called PURLINS (Fig. 2).

2. Types of Wooden Trusses

pe when the span When there is no tie-beam the rods nd 5, merely sup- am and are often

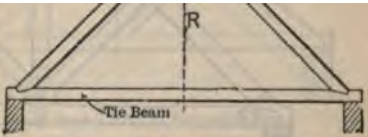


Fig. 1. Simplest Three-piece Truss. Spans up to Twenty-four Feet

Howe Trusses. own in Figs. 4 and mes called Howe character of the stresses in the web-members corresponds with sses in the web in the standard form of Howe truss. They are NGULAR HOWE TRUSSES to distinguish them from the STANDARD ith parallel chords.

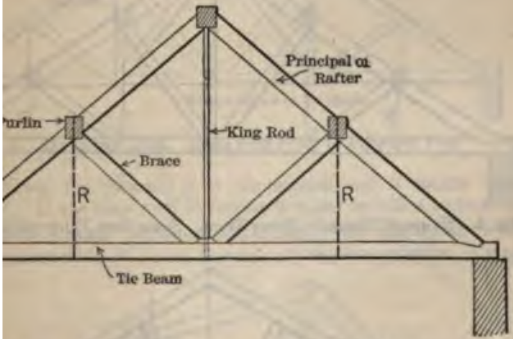


Fig. 2. King-rod Truss. Spans up to Thirty-six Feet

Truss. The RISE of the rafter in any of the trusses, Figs. 1 to

Counters. The truss shown in Fig. 6 differs from those shown i to 5, in not being composed entirely of triangles and in having a re the middle. Assuming the joints to be pin-connected and without b

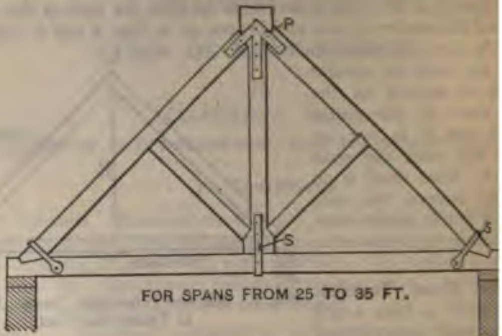


Fig. 3. Modern King-post Truss

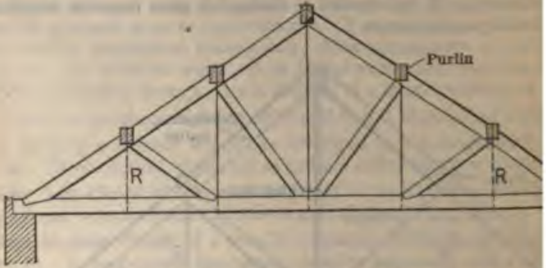


Fig. 4. Six-panel Triangular Howe Truss. Spans from Thirty-six to F

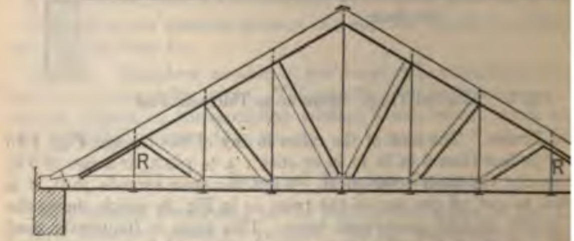
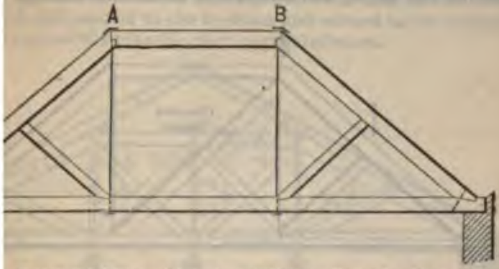


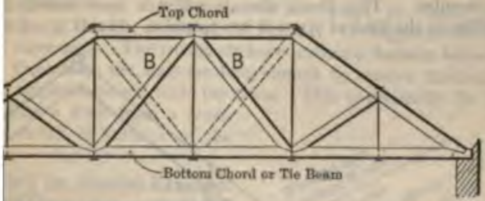
Fig. 5. Eight-panel Triangular Howe Truss. Spans from Forty-eight to

is evident that a very small inequality in the position or magnitude ing will cause the failure of the truss since the rectangle will not retai This is easily verified by means of a cardboard model fastened a elets. When the joints at the corners of the rectangle are not turn they have a tendency to prevent distortion. When the



Queen-rod Truss. Spans from Thirty to Forty-five Feet

y upon the left of the center the truss itself tends to assume a hat shown in Fig. 8. The DISTORTION of the rectangle may be : introduction of a diagonal member as shown in Fig. 9. For



Queen-rod Truss. Spans from Forty to Fifty-two Feet

vn, the diagonal is in compression and is usually called a If the piece were in tension it would be called a COUNTERTIE.

al Loads. Although roof-trusses of the type shown in Fig. 9, etrical loads, do not theoretically require COUNTERS, it is never-

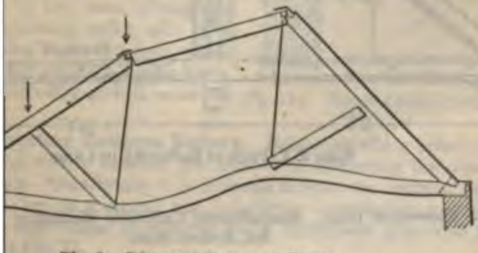


Fig. 8. Distorted Queen-rod Truss

to brace the rectangle along both diagonals to insure stability unsymmetrical loading and to relieve the joints from any latter, which is usually caused by wind, snow and floor-loads.

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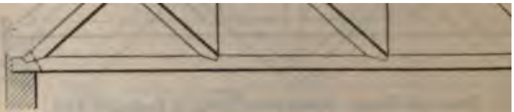
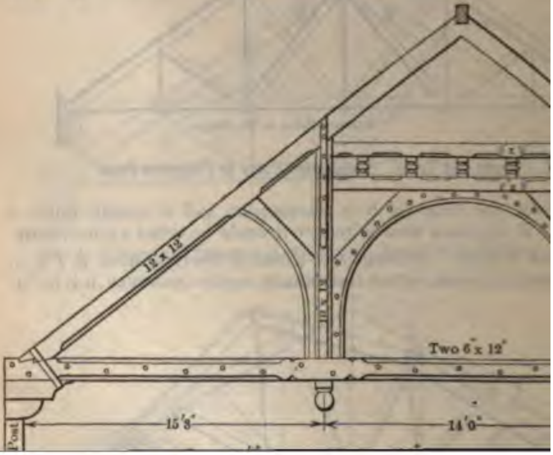
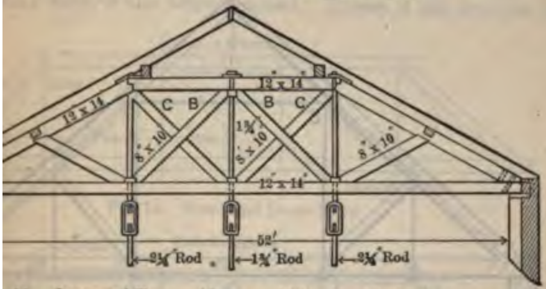


Fig. 9. Counterbraced Queen-rod Truss

is advisable to introduce a member following the other diagonal lateral containing the member subjected to the two kinds of stres main member. This piece, also, is called a COUNTERBRACE according to the kind of stress it has to resist. If this is not don



Mr. William G. Preston, is shown in Fig. 10. The truss-members, long-leaf yellow pine, were worked from timbers of the dimensions his truss wooden members instead of rods are used for the vertical bolted and tenoned to the tie-beam and secured to the rafters by The curved ribs take the place of counterbraces.



11. Queen-rod Truss. Museum of Fine Arts, St. Louis, Mo.

-Rod Truss from the Museum of Fine Arts, St. Louis, Mo., Peabody & Stearns, is shown in Fig. 11. It supports the floor ns of three rods. The truss-rods have nuts and washers below the the threads on the rods are long enough to receive turnbuckles t the suspension-rods with the truss. This is generally the best spending a floor from a truss. ws a detail of joint *A* of the 11.

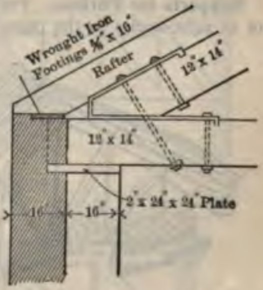


Fig. 11A. Detail of Joint *A*, Fig. 11.

Omitted for Special Reasons. a truss, sometimes used when it keep the middle part of an attic bstructions. In building this dvisable to construct the lower fters of two timbers, thoroughly er, as shown. What has been to counterbraces in queen-rod s also to this truss, although in e continuous rafter aids very resisting distortion from wind- that for ordinary construction not exceeding 40 ft it is safe to races.

f Supporting Common Rafters. Before describing other types may be well to consider the manner of supporting the common trusses. Occasionally it is desirable to span the common rafters truss, but as a general rule it is better construction to support ns of large beams or PURLINS which themselves span from truss own in Fig. 13.

The trusses can be designed so that the purlins need not be more art, and very often not more than 6 or 8 ft apart; so that the rs need not be more than 2 by 4 or 2 by 6 in in cross-section, while ay be spaced 12, 14, or 16 ft on centers. As *a rule a spacing of*

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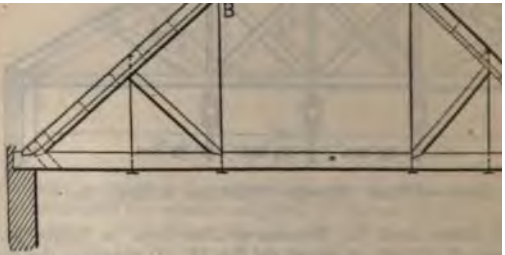
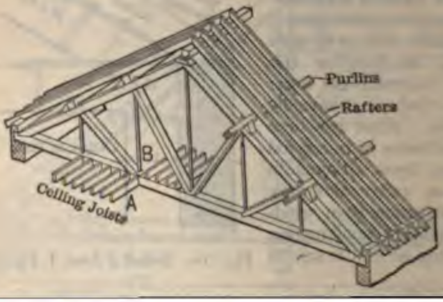


Fig. 12. Queen-rod Truss with Middle Part Clear. Spans up to

they supported the common rafters. For wooden trusses of
span, purlins should always be used.

Supports for Purlins. Purlins may be placed with their s
or at right-angles to the plane of the roof, as shown in Figs



ends
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Fig.

the types shown in Figs. 14 to 17 are undoubtedly the most satis-
wooden construction, when the span does not exceed 80 ft; and
alities where the cost of iron rods is relatively great, it is as econom-
In this work the name HOWE TRUSS is given to this type, as it is
on of the Howe bridge-truss to building-construction; and the
NTAL TRUSS is also sometimes used. Trusses of this type can be

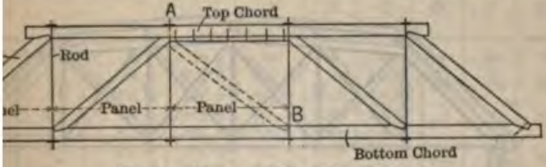


Fig. 14. Five-panel Howe Truss

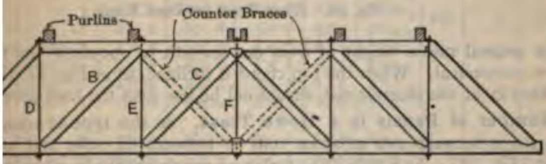


Fig. 15. Six-panel Howe Truss

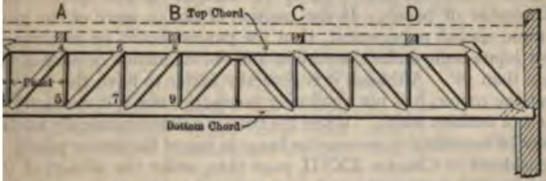


Fig. 16. Ten-panel Howe Truss

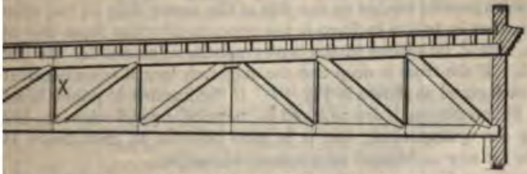


Fig. 17. Six-panel Howe Truss with Top Chord Inclined

enough for spans up to 150 ft; but when the span exceeds 100 ft
y cheaper to use a steel truss of the PRATT TYPE in which the verti-
ompression and the diagonals in tension. When a Howe truss is
longitudinal direction of a flat roof, the top chord may be given
n of the roof itself, so as to support the rafters without the blocking
Fig. 17. For deck roofs the top chord may be inclined upwards
enter or deck-ridge, to conform to the shape of the roof, as shown in

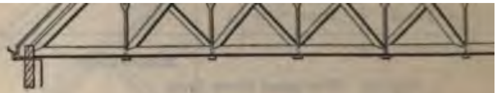


Fig. 18. Howe Truss for Deck Roofs

As a general rule a height of from one-seventh to one-sixth th
most economical. When the top chord is inclined, as in Fig. 17
X, that is, at the shortest rod, should not be less than the limit g

Number of Panels in a Howe Truss. In this type of t
the space between two adjacent rods or between an outer roc
joint (Fig. 14). As a rule, the number of panels should be such
onals will have an inclination of from 36° to 60°, an inclinatio
being the most economical. It is not material whether there is
odd number of panels. If the position of one or more of the
by some special requirement, then the panels should be so arr
upper joints come under the purlins, and the inclination of none o
is less than 36°. Although it is generally better to have the tru
about the center, it is not absolutely necessary; nor is it necessa
panels of uniform width. When the truss is not symmetrically lo
it may be necessary to reverse the brace in one of the center pan
is considered in Chapter XXVII, page 1102, under the subject o
CALLY LOADED TRUSSES.

Counterbraces in a Howe Truss. If there is any chan
being more heavily loaded on one side of the center than on the o
BRACES, that is, braces inclined in the opposite direction from th
braces, should be placed in the center panels, as shown by the
Fig. 15. If the truss is deep and the diagonals long it is econon
brace each panel as shown in Fig. 18. If the number of panels
in Fig. 14, no diagonals are required in the middle panel when th
loading are symmetrical; but it is good practice to cross-bra
provide for any accidental unsymmetrical loading.

imensions for Six-Panel Howe Trusses, Symmetrically Loaded

ber, Norway pine, Douglas fir, or eastern spruce. (See Fig. 15)

Total height		Top chord	Bottom chord	Braces			Rods not upset		
				A	*B*	*C*	*D*	*E*	*F*
ft	in	in	in	in	in	in	in	in	in
6	7	6× 6	6× 8	6× 6	6×4	6×3	1⅛	¾	⅝
5	2	6× 8	6× 8	6× 6	6×6	6×4			
6	8	6× 8	6× 8	6× 6	6×4	6×3	1¼	⅞	⅝
5	2	8× 8	8× 8	8× 6	6×6	6×4			
6	8	6× 8	6× 8	6× 8	6×6	6×4	1¼	⅞	⅝
5	2	8× 8	8× 8	8× 8	6×6	6×4			
7	7	8× 6	8× 8	8× 6	8×4	6×4	1¼	⅞	⅝
5	11	8× 8	8× 8	8× 6	8×5	8×4			
7	8	8× 8	8× 8	8× 6	8×5	6×4	1⅜	⅞	¾
5	11	8× 8	8× 8	8× 8	8×6	8×4			
7	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
6	1	8×10	8×10	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	⅞	¾
6	8	8× 8	8× 8	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	1	¾
6	10	8×10	8×10	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
6	10	8×10	8×10	8×10	8×6	8×4			
9	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	⅞	¾
7	6	8× 8	8×10	8× 8	8×6	8×4			
9	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
7	7	8×10	8×10	8× 8	8×6	8×4			
9	10	8×10	8×10	8×10	8×8	8×6	1⅝	1⅛	¾
7	7	10×10	10×10	10× 8	8×8	8×4			
10	9	8× 8	8×10	8× 8	8×6	6×6	1⅜	1	¾
8	4	8×10	8×10	8×10	8×6	8×4			
10	10	8×10	8×10	8×10	8×6	6×6	1½	1⅛	¾
8	4	10×10	10×10	10× 8	10×6	8×4			
10	10	10×10	10×10	10× 8	10×6	8×6	1¾	1⅛	¾
8	4	10×10	10×10	10×10	10×6	8×6			
12	6	8×10	8×10	8×10	8×6	6×6	1½	1	¾
9	7	10×10	10×10	10× 8	10×6	8×6			
12	6	10×10	10×10	10× 8	10×6	8×6	1¾	1⅛	¾
9	9	10×12	10×12	10×10	10×8	10×6			
12	6	10×10	10×10	10×10	10×6	8×6	1⅞	1¼	⅞
9	9	10×12	10×12	10×12	10×8	10×6			
14	2	10×10	10×10	10×10	10×6	8×6	1⅝	1⅛	¾
10	10	10×10	10×10	10×10	10×6	8×6			
14	2	10×10	10×10	10×10	10×8	8×6	1⅞	1¼	⅞
11	0	10×12	10×12	10×10	10×8	10×6			
14	4	10×12	10×12	10×12	10×8	8×6	2	1⅜	1
11	1	10×12	10×14	10×12	10×8	10×6			

tables giving the proper dimensions of the members of such gives the dimensions for six-panel trusses for heights of one-sixt the span and for three different spacings. These dimension roof covered with tin, sheet iron, or composition; a snow-load o equivalent to about 24 in of light, dry snow; also for a lath-ar supported by the bottom chord. The chords and braces are and the rods of wrought iron. These dimensions apply only

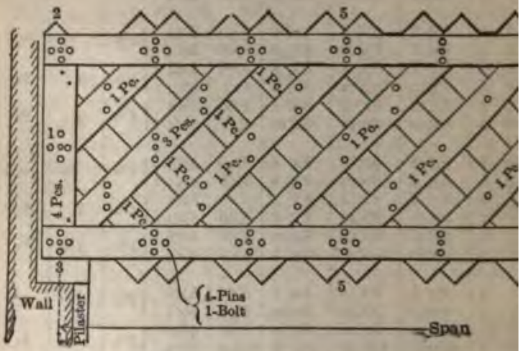
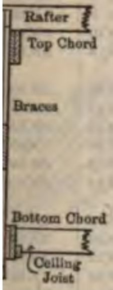


Fig. 19. Lattice Truss

are supported on purlins placed at the upper joints, as in When the rafters rest on the top chord, as in Fig. 17, th the latter must be increased and special calculations made for sions given in the table may be used for trusses of greater ges have been constructed with trusses of this type, some of which e now (1915) in New England. The principal objections to the tendency to twist sidewise, like a thin board on edge, its flexibility plane and the difficulty of getting sufficient bearing material at As indicated in Fig. 19, the truss is composed of top and bottom lly parallel, connected by lattice bracing. The chords are com- planks, two being on one side and two on the opposite side of the ie bottom chord the planks should be as long as can be obtained t so that no two splices are near the same point. The available ottom chord to resist tension is the area of three planks less the area e joints by the connecting pins or bolts. Each member of the web single plank arranged as shown in Fig. 19. The braces are inclined of about 45° and usually three sets are sufficient, as shown in the connections are best made with American-locust pins, which give areas without much extra weight. Modern construction employs are expensive and add considerable weight. There should be at



rtical Section of
wn in Fig. 19

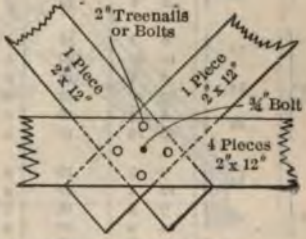


Fig. 21. Lower Joint 5 of Truss Shown in Fig. 19

s at each connection, if the planks are wide enough to permit, and t, at the chord-joints. Since about one-half the web planks resist es, the web projects beyond the chord at least 4 in to provide gitudinal shearing area. The ends are reinforced by vertical tim- etween the chords and each set of diagonals is thoroughly fastened ers. In some cases it is necessary to add timbers on the outside tend them down to the lower face of the bottom chords to relieve sive bearing-stresses where they rest on the supports. The methods ig the stresses in this truss are considered in Chapter XXVII, 1091. Figs. 20 and 21 show details of this lattice truss.

Trusses with Raised Bottom Chords. All of the trusses thus have horizontal bottom chords; and this construction is the most vell as the most economical and should be used whenever condi- necessitate a greater height of ceiling. In roofing churches, public ised ceilings are often desirable as they increase the general height hout increasing the height of its side walls.

Table II. Dimensions for Lattice Trusses, Uniformly

Timber, Norway pine, Douglas fir, and yellow pine. (Fig.

Span	Spacing of trusses	Height out to out of chords		No. of spaces	No. and size of pcs of bottom chord	No. and size of pcs of top chord	Size of braces
ft	ft	ft	in		in in	in in	in in
40	12	5	6	16	4 2×6	4 2×6	2×6
		7	2	12	4 2×6	4 2×6	2×6
	14	5	7	16	4 2×6	4 2×8	2×6
		7	3	12	4 2×6	4 2×8	2×6
	16	5	8	16	4 2×8	4 2×8	2×8
		7	4	12	4 2×8	4 2×8	2×8
50	12	6	8	16	4 2×8	4 2×8	2×10
		8	8	12	4 2×8	4 2×8	2×10
	14	6	8	16	4 2×8	4 2×8	2×10
		8	8	12	4 2×8	4 2×8	2×10
	16	6	9	16	4 2×8	4 2×10	2×10
		8	8	12	4 2×8	4 2×8	2×10
60	12	8	4	16	4 2×10	4 2×10	2×10
		10	10	12	4 2×10	4 2×10	2×10
	14	8	4	16	4 2×10	4 2×10	2×10
		10	10	12	4 2×10	4 2×10	2×10
	16	8	4	16	4 2×10	4 2×10	2×10
		10	10	12	4 2×10	4 2×10	2×10
70	14	9	5	16	4 2×10	4 2×12	2×10
		12	4	12	4 2×10	4 2×10	2×10
	16	9	5	16	4 2×10	4 2×12	2×10
		12	4	12	4 2×10	4 2×10	2×10
	18	9	6	16	4 2×12	4 2×12	2×10
		12	6	12	4 2×12	4 2×12	2×10
80	14	11	0	16	4 2×12	4 2×12	2×12
		14	0	12	4 2×12	4 2×12	2×12
	16	11	2	16	4 2×14	4 2×14	2×12
		14	0	12	4 2×12	4 2×12	2×12
	18	11	2	16	4 2×14	4 2×14	2×12
		14	1	12	4 2×12	4 2×14	2×12

Note. All joints should be thoroughly spiked and packing blocks us
sary. When treenails are used each chord-joint should have in add
bolt as shown in Fig. 21.

Scissors Trusses. For the roofs described in the preceding pa
form of the SCISSORS TRUSS, so named from its resemblance to
sors, is most often used. When correctly designed, with members
size, and with joints carefully proportioned to the stresses, it is
truss for supporting roofs over halls and churches, up to a span
for greater spans it should be used with caution, as the stresses
great and the joints difficult to make. Figs. 22 to 27 show diffe
this truss and modifications of it adapted to different spans and
None of these trusses exerts a large horizontal thrust if the m
ample size and the joints properly made. The members having

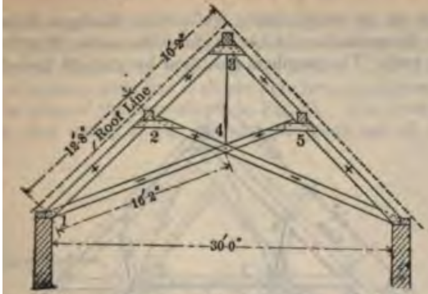


Fig. 22. Simple Scissors Truss. Spans up to Thirty Feet

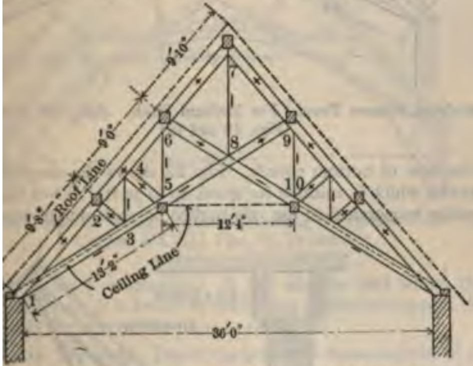
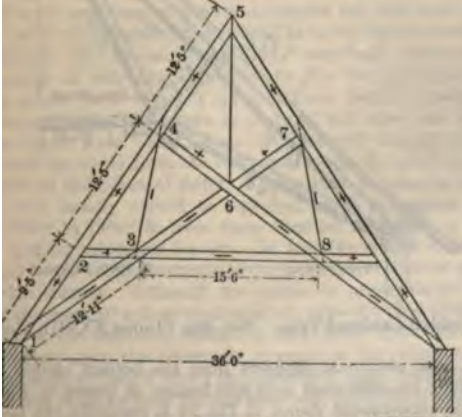


Fig. 23. Scissors Truss. Spans Exceeding Thirty Feet



ssors Truss. For Steep Roofs. (See Chapter *XXVIII, Figs. 18 and 19*) or close to them are in COMPRESSION, while those having a minus TENSION. The determination of the actual HORIZONTAL THRUST is on pages 1085–1087. The members indicated by a single line should

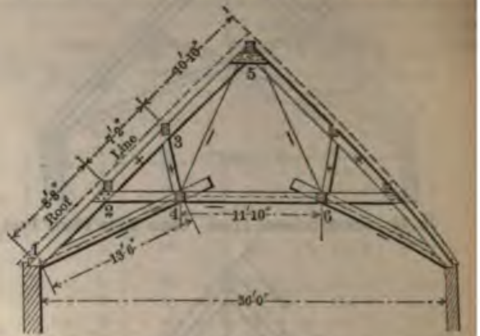


Fig. 25. Modified Scissors Truss. For Medium Pitch. (See, also, Chapter Figs. 18 and 19)

except in the case of bottom chords. Fig. 22 shows the simplest SCISSORS TRUSS, which is suitable for spans up to 30 ft. When the 30 ft, it is more economical to use two purlins on each side to support

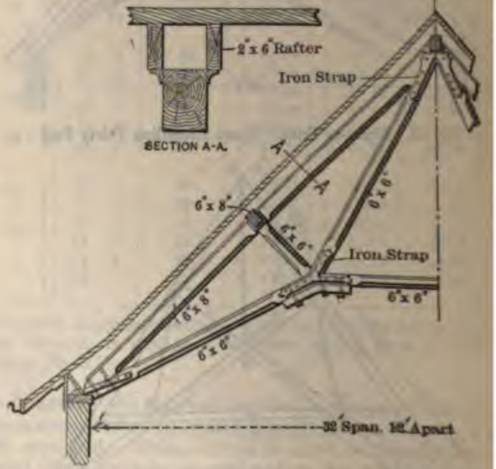
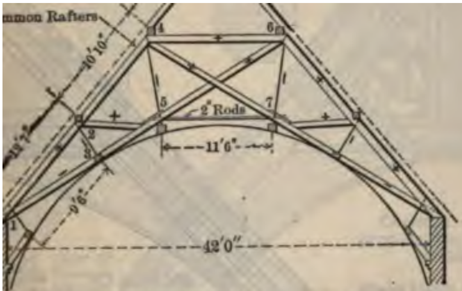


Fig. 26. Finished Cambered Truss. (See, also, Chapter XXVIII, Figs.

mon rafters; and additional supports from the bottom chords required, calling for additional rods and braces, as shown in Fig. steep roof the arrangement shown in Fig 24 is generally the best



ied Scissors Truss. Spans Exceeding Thirty-six Feet. (See, also, Chapter XXVIII, Figs. 18, 19 and 20)

t, is more economical. It can also be used where the roof is this form it is better to use ceiling-purlins to support the ceiling-span the latter from truss to truss.

eam Trusses. Two of the principal characteristics of the Gothic tecture are the relatively elaborate ornamentation of structural exposure to view of the construction of a building as a whole. d arch and steep roof were developed the roof-truss became an ure in the ornamentation as well as in the construction of Gothic rches. The trusses of this period were built almost entirely of erally of very heavy timbers, to give the appearance of great e of the most common types of these Gothic trusses, and also mental, was the HAMMER-BEAM truss, still often used in churches e Gothic style. Figs. 28 and 29 show early English forms of this akes its name from the horizontal beam *H*, called the HAMMER-oot of the principal rafter. In the more ornamental trusses this was usually carved to represent royal personages or angels.

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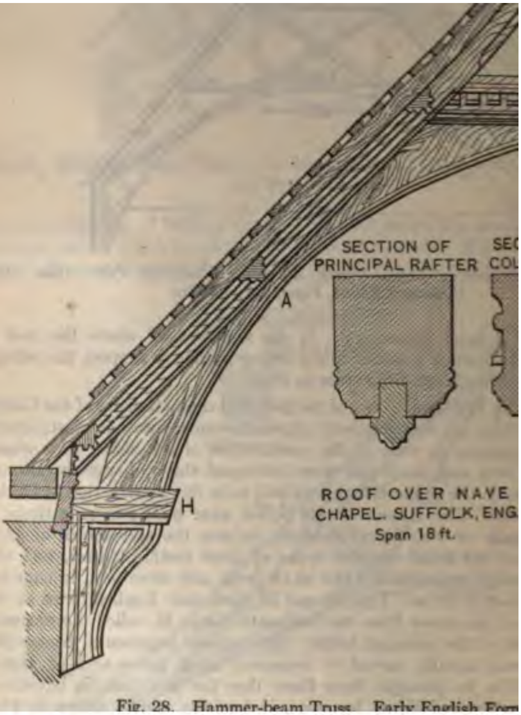

Fig. 28. Hammer-beam Truss. Early English For

First Church, Boston, Mass. An excellent example of a truss adapted to modern conditions is shown in Fig. 30, which e-half of one of the trusses designed by Ware & Van Brunt, for

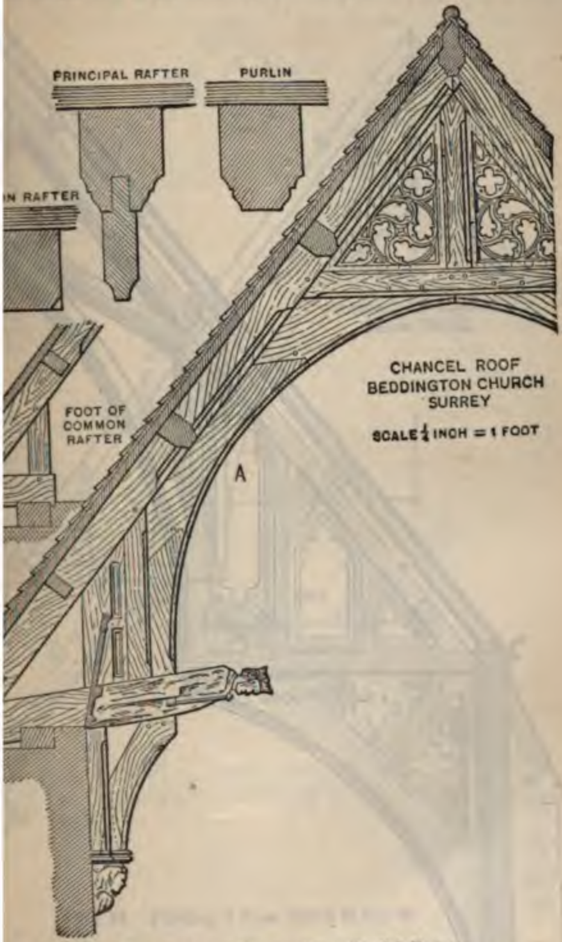

Fig. 29. Hammer-beam Truss. Early English Form

hurch, Boston, Mass. The truss is finished in black walnut and has f being very strong and heavy. Fig. 31 shows the framing of the without the casing and FALSEWORK. It should be noticed that urned column in the upper part of the truss, Fig. 30, there is an iron

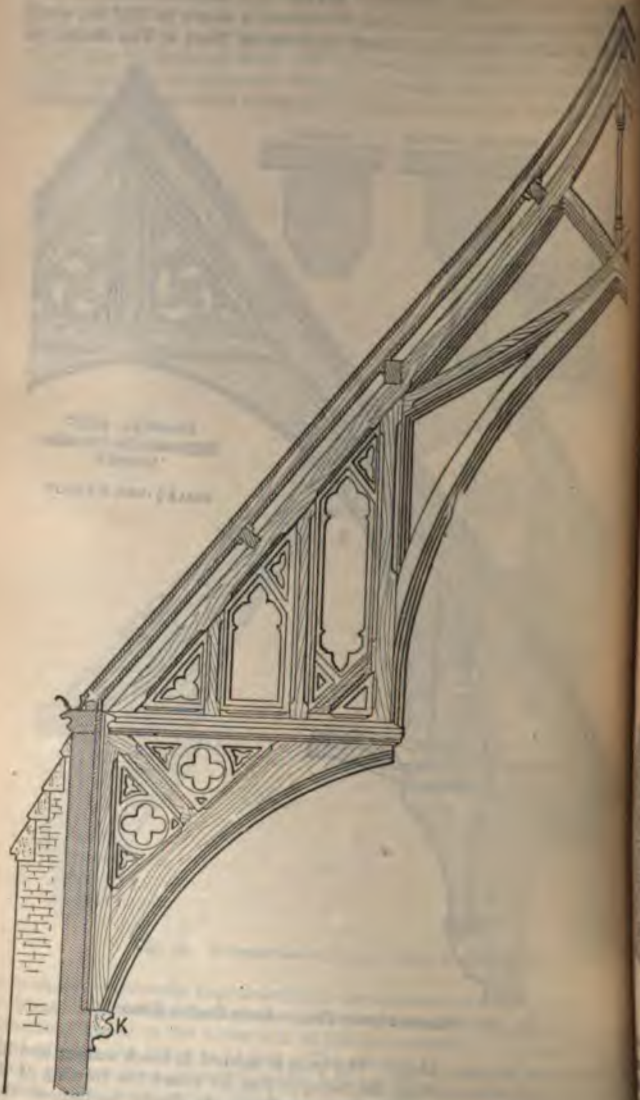

Fig. 30. Hammer-beam Truss. First Church, Boston, Mass.

which resists the tensile stress. In this form of truss the line of st of the arch enters the wall just above the CORBEL, *K*; and, as s inclined only about 30° from the vertical, its tendency to overll is not very great, and may be resisted, in this particular case,

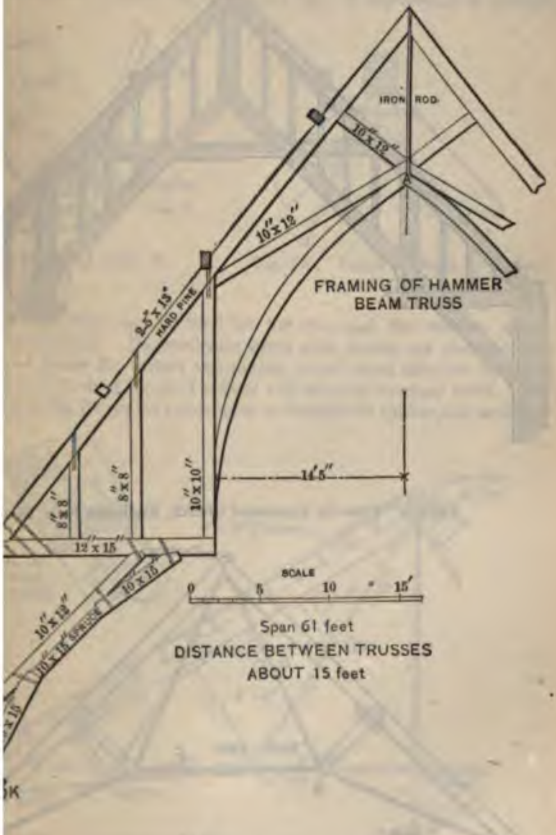

Fig. 31. Framing of Truss Shown in Fig. 30

in or 2 ft thick, thoroughly reinforced by a BUTTRESS of proper uilt on the outside. In trusses of this kind, the various members urely fastened together wherever they cross or touch each other, ture as a whole made as rigid as possible. No dependence should on the casings and panel-work for any extra strength.

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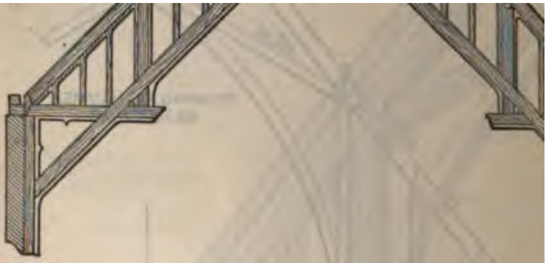
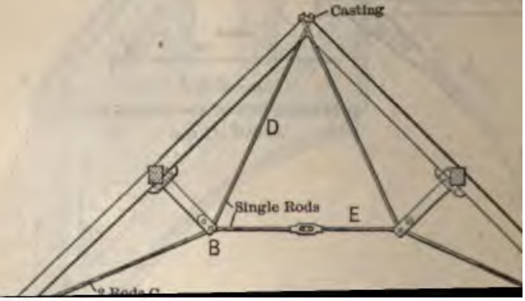
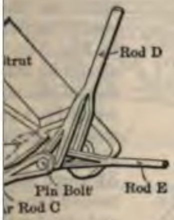


Fig. 32. Truss for Emmanuel Church, Shelburne Falls, Ma



els. The action of the stresses in hammer-beam trusses is pter XXVII, pages 1087 to 1089.

sses with Iron Ties. Where there is no ceiling beneath the sirable to make the trusses as light in appearance as possible,



Detail of Joint *B*, ig. 33

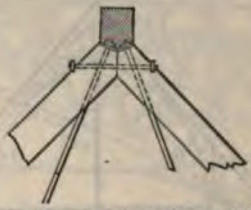


Fig. 33B. Detail of Joint at Ridge, Fig. 33

steel rods may be used for the ties, and the wooden rafter- retained. For moderate spans such trusses are cheaper than d where the rafters and purlins are of wood they are about as ind 34 show forms of trusses well adapted to many roofs. The in Fig. 34 are for yellow-pine or Douglas-fir timber and wrought-

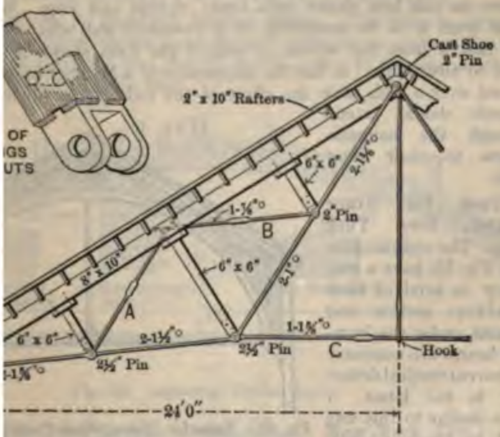


Fig. 34. Wooden Truss with Iron Ties

e ample for a slate roof, the trusses being spaced from 12 to . Trusses of the form shown *in Fig. 33 are sometimes made and D continuous. They should not be made in this way, he entire rod is proportioned for the stress in C, as this stress in D. The best construction for the joint B* is illustrated

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in Fig. 33A, which shows a CAST-IRON SHOE fitted to the end of the
receive the pin. For the truss shown in Fig. 34, a shoe made as show
detail drawing makes a better connection for the rods, two of the latt
placed outside of the brackets and three between them. For a trus
single strut, a TURNBUCKLE on the rod *E* serves to tighten the rods
there are three stru
should be five turnb
in Fig. 34. A cast-i
should be made to re
foot of the rafter and
secured to a pin passe
shoe and rafter. At
also, of the truss sho
34, there should be
receive the ends of t
and pins for the tie-
apex-joint of the
33) may be made
crossing the rods
CAST WASHER, or as
Fig. 33B. The pi
joints should be co
shear, bearing an
More modern const
places the cast iron shown with STEEL PLATES and PINS. When
beam truss is to be supported on a clerestory-wall which is not
nor braced from the outside, a truss of the form shown in Fig.
used to advantage. It has the appearance of a hammer-beam trus
placed over a high nave the effect of the rods is not objectiona
tie-rods should extend through the hammer-beams to their outer ends.

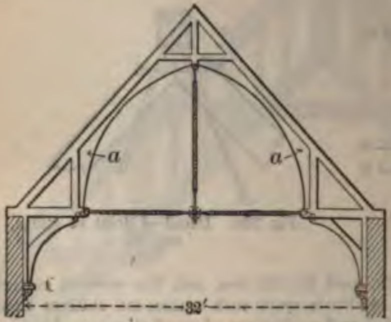


Fig. 35. Hammer-beam Truss for Grace Chapel, New York City

Truss for Grace Chapel, New York City. The CURVED RIBS *a*, *a*, Fig. 35, have a tendency to bend at their smallest section and BRACES under the hammer-beams are necessary to prevent vertical deflection in the latter. A truss similar to this was used in Grace Chapel, New York City.

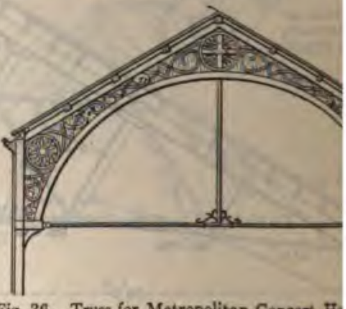


Fig. 36. Truss for Metropolitan Concert H
City

Truss for Metropolitan Concert-Hall, New York City. Fi
a form of truss used to support the roof of the Metropolitan Conce
York City, George B. Post, architect. The span is about 54 ft and
tions are about as shown. The arch between rafters and raised
mented with sawed work and the truss has a very light and airy
The tie-rod is kept from sagging by a vertical rod from the
arch.

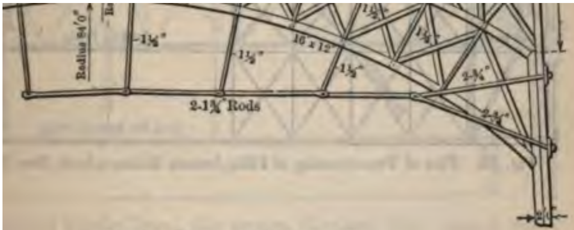
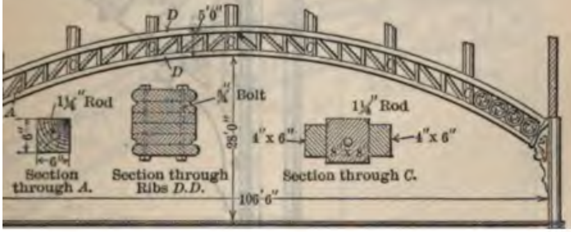


Fig. 37. Segmental Timber Arch

CHED RIBS supporting all the load and the TIE-RODS preventing the rch from the spreading which would result without them.

M. C. M. A. Building, Boston, Mass. This truss is shown in the framework shown above the arch is simply to support the purers and carry the load directly to the arch. It does not assist the way in carrying the load.



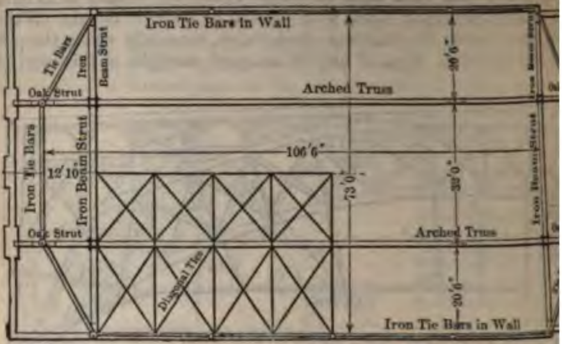


Fig. 39. Plan of Truss-framing of Fifth Avenue Riding-school, New Yor

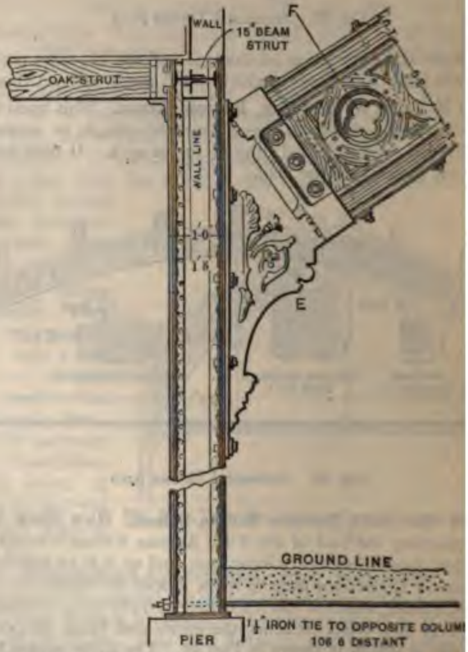
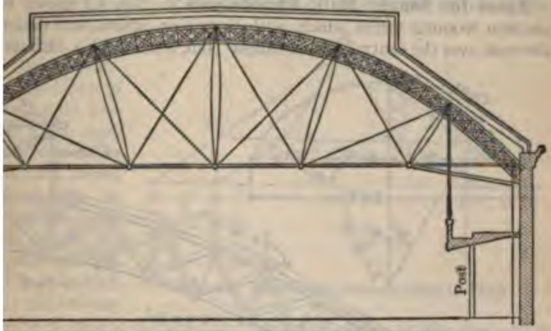


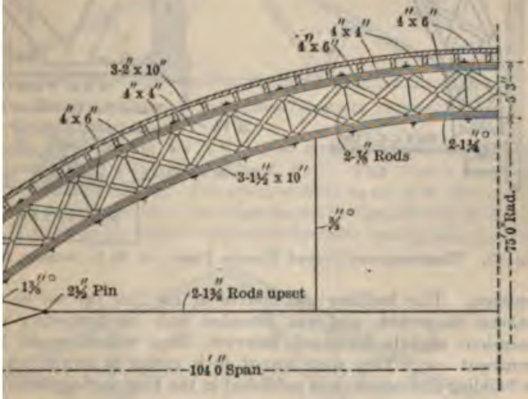
Fig 40. Detail of Iron Skewback and Post of Truss Shown in F

ying a roof-area, equal to about 2 930 sq ft, and a great amount work. The method employed to resist the thrust of these large the use of rods showing in the room is very ingenious. Opposite of the iron posts which receive the arched ribs are oak struts



hed Wooden Truss. City Armory, Cleveland, Ohio. Span 79 feet

y iron tie-bars and heavy iron beams and together forming a at each end. These two trusses are prevented from being pushed 1-in iron tie-bars in each side wall, as shown in the plan (Fig. 39). of the two iron posts are tied together by iron rods running under



42. Arched Wooden Truss, Sänger Hall, Philadelphia, Pa.

hole length of the room. Altogether this gives for the tie-rods wo 3 by 1-in iron bars and one 1½-in-diam iron rod, equivalent -in tie-bars. Enlarged sections of the ribs, uprights and braces ig. 38. It should be noticed that the uprights have iron rods xes, holding the two ribs together. *Fig. 40 shows a detail,* or

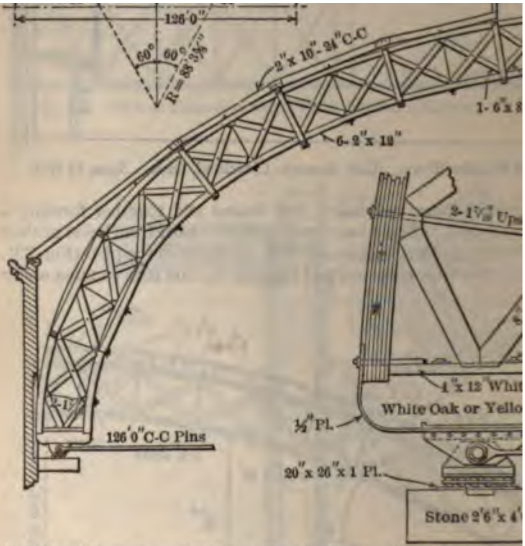


Fig. 43. Three-centered Curved Wooden Truss. O. N. G. Armory

architects. This building was erected in 1897 for the use
National Sängerfest, and was intended only for temporar
dimensions slightly increased, however, these trusses woul

lso much more
one dimension
loes not exceed
they can be
eted up in the
ase they are
judgment will
to parts which
by rail, leaving

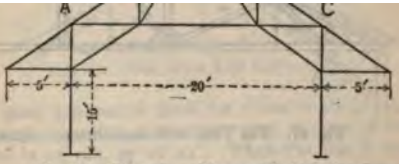
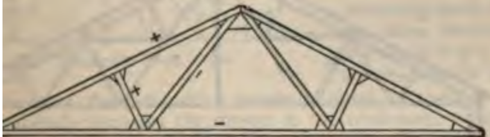


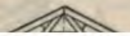
Fig. 44. Truss for a Narrow Shed or Shop

to be riveted at the building; but entire trusses having spans even
be raised from the ground and put in place. Occasionally a struc-
h magnitude that this is not feasible, in which case the trusses
l in parts and riveted afterwards. For a narrow shed or shop a



45. Simple Fink Truss. Spans from Twenty to Thirty-six Feet

hape shown in Fig. 44 is the most economical, the truss proper
rtion enclosed within the points *A*, *B*, *C*. This truss is practically
hat shown in Fig. 45. For spans of from 24 to 48 ft, and inclina-
eding 6 in to the foot, the types shown in Figs. 46 and 47 are the
Trusses of the types represented by these two figures are called



one of the forms shown in Figs. 48, 49, 50, or 51 should be used. FRENCH appears to be generally given to those trusses in which the raised or cambered in the middle. The truss shown in Fig. 51 m

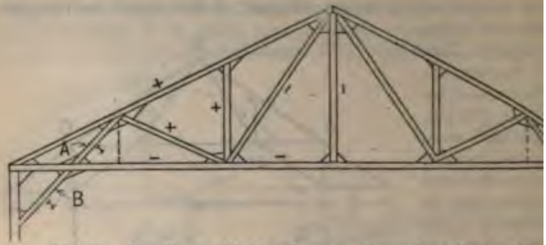


Fig. 47. Fan Truss with Knee-braces. Spans from Forty to Sixt

a TRIANGULAR PRATT TRUSS as the web is composed of verticals in and diagonals in tension. This truss is not as economical as the except when the inclination of the rafter is less than ¼ pitch. account of the great length of the web-members in compression.

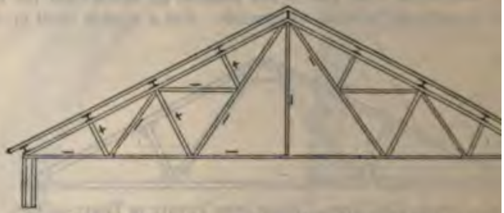


Fig. 48. Fink Truss. Spans from Forty to Eighty Feet

steel trusses it is desirable to have as many members, and espec long members, in tension as possible, as a given weight of steel greater stress when in tension than when in compression. The of FINK TRUSSES and FAN TRUSSES lies in the fact that most of

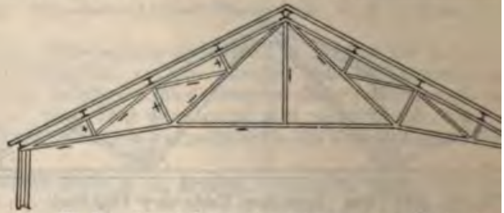


Fig. 49. French Truss. Spans from Forty to Eighty Fee

are in tension and the struts are short. By comparing Figs. 5 seen that the inner strut in Fig. 50 is only one-half as long as th 51. If the roof is hipped it is desirable to have vertical memb trusses to receive the short trusses or TRUSSED PURLINS.

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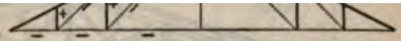
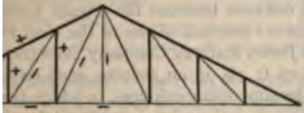


Fig. 50. Fink Truss with Vertical Struts

be used on a
ng ⅝ in to the
lering the con-
the roof and
the trusses, the most economical pitch for a roof is about one-
an, or what is commonly called a QUARTER-PITCH, the rise of the
6 in for each 12 in of run, or 26° 34′. When the rise is less
th the span some other type of truss is generally required.
clination of the roof is determined almost entirely by the question
he rise is generally made from 6 to 7 in in 12 in. With FINK TRUSSES or FAN TRUSSES having inclinations for the rafters not exceeding 30°, it is more economical to employ a horizontal chord or tie. A truss whose bottom chord has a rise of 2 or 3 ft, as in Fig. 49, presents a better
however, than one with a horizontal chord. Raising the bottom
aterially increases the stresses in the truss-members and hence
cost. For steep roofs, however, it is generally as economical to
tom chord, because of the shortening of the members.



. 51. Triangular Pratt Truss

f Panels. The NUMBER OF PANELS that should be used in each
uss is determined in great measure by the construction of the roof.

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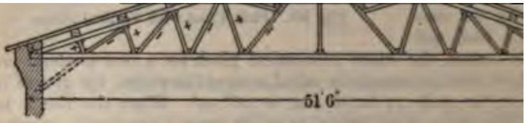
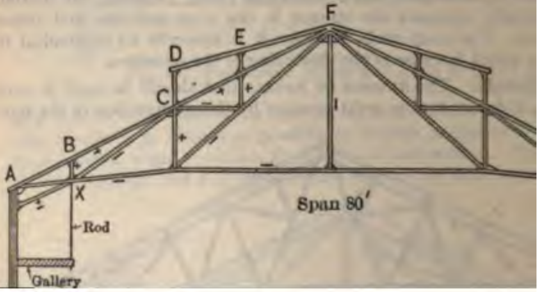


Fig. 53. Fink Truss. Span Fifty-one Feet Six Inche

the distance between the latter should not exceed 9 ft, and pref
depending somewhat upon the distance between the trusses th
diagram shown in Fig. 55 represents one-half of one of the steel
roofing a car-barn for the North Jersey Railway Company, Newar
are 13 of these trusses spaced 19 ft 2¼ in on centers, each ha
98¼ ft between the centers of the supporting columns, to which t
by splice-plates engaging the end connection-plates and the webs
The dimensions of the principal members of these trusses are in



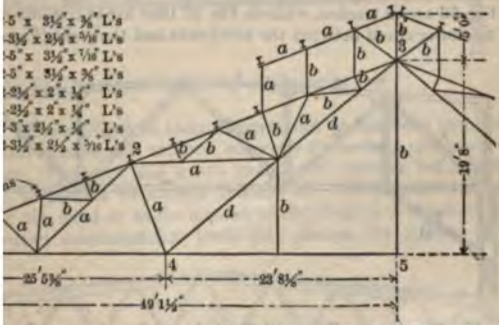
end. The general spacing of the purlins is 4 ft 9¾ in. This of an extremely light roof, the weight of each truss being about entire weight of truss, purlins, bracing of lower chord and ofing being only 8 lb for each horizontal foot of surface cov-

ist of Descriptions of Different Types of Roof-Trusses

Engineering Record

Date	Type	Number of panels
9, 1892	Howe	8
1901	Fink Fan	8 12
4, 1902	Fink	8
y 22, 1902	Pratt	6
12, 1905	Fan	12
ber 2, 1905	Fan	12
ber 16, 1905	Fink	8
ber 2, 1907	Truss	16
ber 16, 1911	Truss	12
7, 1911	Fink	16

ll-hall. The truss shown in Fig. 54 was designed for the roof ng a span of 80 ft and a spacing between trusses of 20 ft. The nstructed with 2 by 8-in rafters supported by purlins at the



er Car-barn, Newark, N. J. Span Ninety-eight Feet, Three Inches.
(See, also, Chapter XXVIII, Fig. 25)

, *E* and *F*. Sashes were to be placed in the rise *CD*, to light building. The joint at *X* was located with reference to the lery-rod; but if there had been no gallery it would have been to space the vertical struts uniformly, as in Fig. 50. In all ated the PLUS SIGN *adjacent to a member denotes that the RESSION, while the MINUS SIGN denotes that it is in* TENSION. *the main rafter, as CD, DE and EF*, in Fig. 54, and *a* and *b n a part of the truss proper, but are merely a* framework to not exceeding 1 in to the foot, one of the types shown in Figs. 5

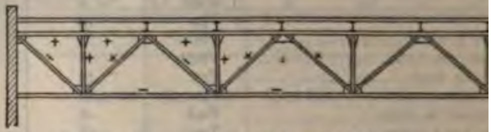
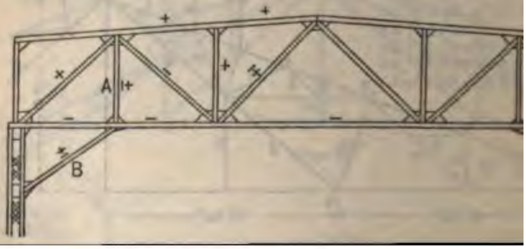


Fig. 56. Warren Truss with Verticals. Span Fifty-six

erally be found economical, the choice of the particular type depe
on the span and on whether the truss is supported by column
stone walls. For spans up to about 50 ft, either of the forms sh
or 57 answer all practical requirements. The truss shown in Fi
to be used where the slope of the roof is at right-angles to the t
built, however, with the top chord inclined as in Fig. 57. Th
in Fig. 56 are, in tension, while in Fig. 57 they are in compression
of the lower chord between the end-joints and the walls (Fig. 56



ıg especially suitable for roofs supported by steel columns. Fig. 58 rom a truss in actual use. The member in the middle indicated by the : should never be omitted, although examples may be found where it een included. Fig. 59, also, represents a roof-truss which was con- vith a span of 57 ft and supported by steel columns. The entire load ıss is transmitted to the columns at the intersection of the diagonals

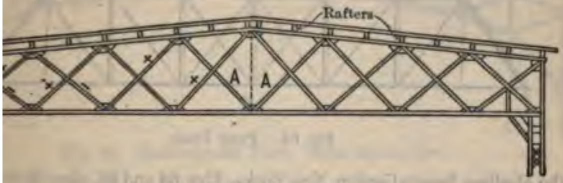


Fig. 58. Double Warren Truss

he top chord. Fig. 60 shows a truss of 96-ft span over a pier-shed, : City, the trusses being spaced 20 ft apart. They are about 10 ft veigh 1 300 lb each. They were delivered from the shops completely and riveted, and were raised and set in position by falls suspended masts. The dimensions of these trusses are given in the Engineering January 18, 1896.

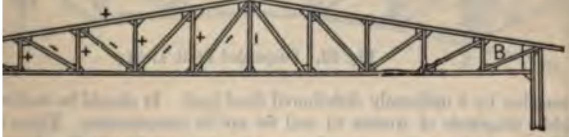
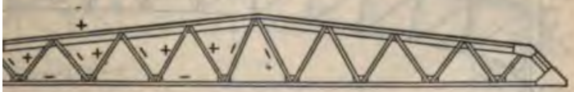


Fig. 59. Pratt-truss Type. Span Fifty-seven Feet

us and Minus Signs in these illustrations, as has been mentioned licate COMPRESSION and TENSION, respectively, under a uniformly dis- ead load. The PLUS and MINUS SIGNS used together indicate that er may be subject to EITHER TENSION OR COMPRESSION according ection of the wind or to the manner of distribution of the snow. In hese trusses unsymmetrical loads may change the stresses in the



Warren-truss Type. Pier-shed, New York City. Span Ninety-six Feet

near the middle of the truss. This CHANGING OF STRESSES due to ading is considered on pages 1096 to 1104. The trusses shown in ɔ 60 are almost invariably built with riveted connections and with annel-shapes for all members.

***tt Truss,** shown in Figs. 61 and 62, is the form of* STEEL TRUSS */ to support floor-loads, the members indicated by double lines* RESSION *and those indicated by single lines in* TENSION. When

Fig. 61. Pratt Truss

of the Madison Square Garden, New York. Figs. 64 and 66, also, of this type, differing, however, from the latter in having all t each half-truss inclined in the same direction. In the typical trus is usually reversed at about the middle of each half-span in ord in tension. The PLUS AND MINUS SIGNS indicate the kind of str

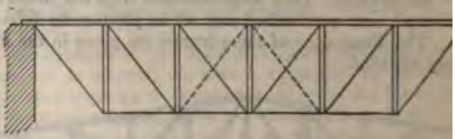


Fig. 62. Suspended Pratt Truss

a member by a uniformly distributed dead load. It should be r middle diagonals of trusses 64 and 66 are in compression. Th well adapted to steel construction and to spans up to 180 ft. exceeds 100 ft one end of the truss should be supported on ROLL the EXPANSION OR CONTRACTION in the steel. In these trusses th

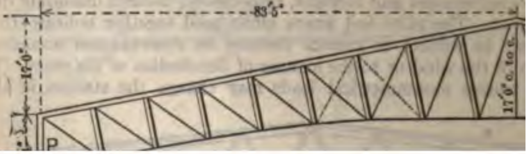


Fig. 64. Quadrangular Truss. Span Eighty Feet

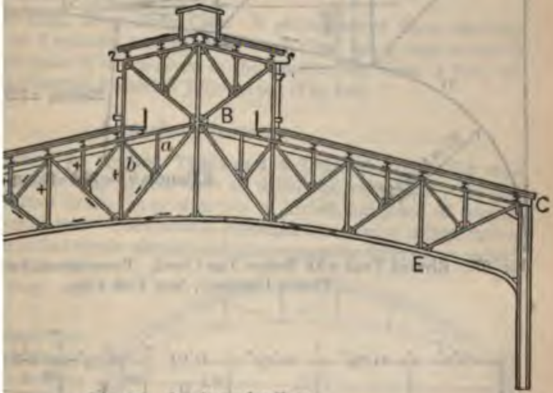
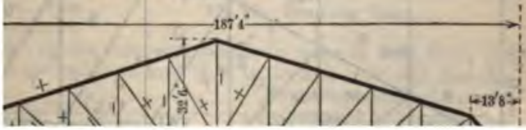


Fig. 65. Quadrangular Truss



chord may be made in the shape of a semicircle or half-ellipse so as t
more of an arch-effect. There are numerous examples in this country of

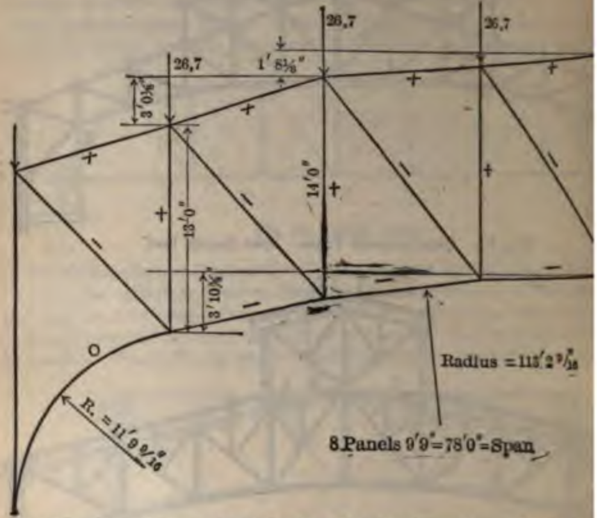


Fig. 67. Riveted Truss with Broken Top Chord, Power-house, Interborou
Transit Company, New York City

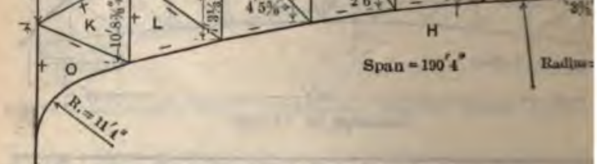


Fig. 68. Pin-connected Truss Over Drill-hall, 71st Regiment Armory, New

rangular trusses having spans of from 100 to 180 ft. For the wider
customary to build the trusses with PIN-CONNECTIONS, EYE-BARS b
the ties. When this is done it is usually necessary to insert

on each side of the truss as shown by the dotted lines, Fig. 63,
nsymmetrical or wind-load the stresses in the diagonals are gener-
For spans less than 100 ft, the trusses may be built with RIVETED
In this case the diagonals are generally made of angles capable
oth tension and compression, the counterbraces, therefore, not
l. For this type of truss the stresses due to wind and snow should
independently of the dead load and the members computed for
stresses produced by every possible combination of loading.

or the Auditorium, Kansas City, Mo. A description, with
f the truss shown in Fig. 66, which is a diagram of one of the trusses
as City Auditorium, may be found in the Engineering Record for
and in the Engineering News of November 2, 1899.

russ with Broken Top Chord. A description is given in the
Record of October 15, 1904. The span is 78 ft between centers
ting columns (Fig. 67).

Drill-Hall, New York City. A pin-connected truss, over the
he 71st Regiment Armory, New York City, has a span of 190 ft
descriptions of it are given in the Engineering News of June 16,
he Engineering Record of July 2, 1904 (Fig. 68).

4. Arched Trusses

Between an Arched Truss and a Trussed Arch. For sup-
oof of very large spaces such as drill-halls, riding-halls, railway
tc., trusses in the form of arches or arches composed of trussed
ften employed. The essential difference between an ARCHED TRUSS
ARCH is that under vertical loads the supporting forces of an arched
tical, while for
rch they are

Trusses. Pre-
year 1880 most
ht-iron trusses
were built in
a bow, from
rm BOWSTRING
Trusses of this type were built with spans of from 88 to 211
rise at the middle of from 1/8 to 1/4 the span. At that time this
type was considered the most
economical for spans exceed-
ing 120 ft, but in recent
years they have been com-
paratively little used. Fig.
69 is the diagram of a bow-
string truss with a span of
153 ft 6 in. The trusses in
this particular case are
spaced 21 ft 6 in apart.
op chord consists of a wrought-iron deck-beam 9 in deep, with
-in plate, riveted to its upper flange. Towards the springing
rengthened *with 7 by 5/8-in plates riveted on each side of the*
The struts are wrought-iron I beams 7 in deep. The bottom
ctional area of 6½ sq in and each diagonal tension-rod a

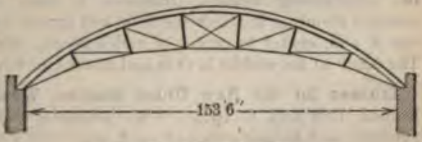
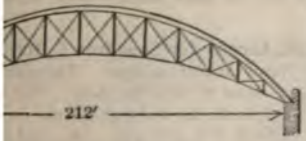


Fig. 69. Bowstring Truss



ig. 70. Bowstring Truss

they are TRUE ARCHES. Probably the frames act partially
and partially as arches.

Trusses for the Conservatory Building, Garfield Par
Engineering News, August 27, 1908. The roof is supported by
spaced 12 ft 6 in on centers. The truss-span is 80 ft 6 in,
end-supports. The chords of the trusses are parallel and conn
BRACING. Both ends of the trusses are bolted to the supports
there must be some horizontal thrust under certain conditio
are riveted at all joints and have no HINGES or PINS.

Trusses for the Chicago and North Western Railway S
Ill. Engineering Record, June 18, 1910. The roof over the
is carried by trusses each having a span of 90 ft 4 in and a rise
riveted to columns about 27 ft 6 in apart. All connections
clear height of the bottom chords at the middle is 84 ft.

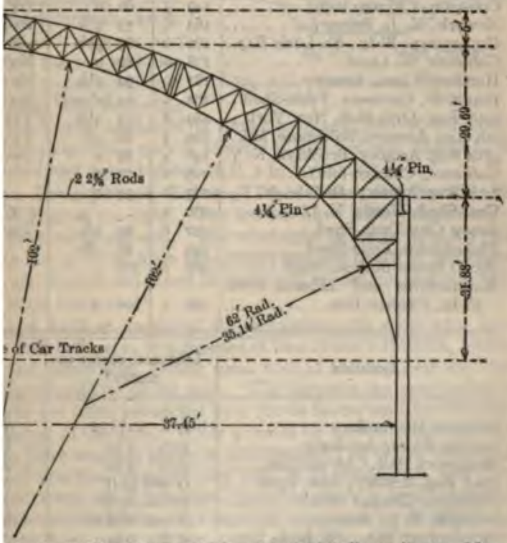
Trusses for the Peoria and Pekin Union Railway Trai
Ill. Engineering Record, December 8, 1900. The trusse
columns about 30 ft above the floor and spaced 20 ft apart.
109 ft 4 in, center to center of end-supports, with a clear ri
The depth at the middle is 18 ft and at the end 6 ft. All conne

Trusses for the New Union Station, Washington, D.
Record, February 6, 1904. The concourse-roof is support
TRUSSES, each having a span of 132 ft 5½ in and a clear rise of
are spaced about 39 ft 4 in apart. One end of each truss re
and the other is riveted to a heavy plate girder. All conne
The bottom chord at the middle is 45 ft above the floor. Th
waiting-room of the same station have a span of 137 ft 8 in
5 in. The chords are parallel and the ends are anchored
masonry.

mns and are spaced from 10 ft 3 in to 19 ft 6 in apart. All ns are made with pins.

s with Horizontal Ties. CURVED OR ARCHED TRUSSES are vith a horizontal member connecting the ends at the supports. ructure as a whole, including the horizontal member, usually , a SIMPLE TRUSS requiring only vertical supporting forces provided one end is free to move, as it is when placed on trusses are supported by long columns it may be assumed e freedom. A few examples are given, some of which are as TRUE ARCHES.

e Sullivan Square Station, Elevated Railway, Boston, ng Record, June 15, 1901. Fig. 71. These ARCHES spring



'russ for Sullivan Square Station, Elevated Railway, Boston, Mass.

and are provided with tension-rods which take up the thrust. sts on two 4¼-in pins at each end, as indicated in the diagram, connected to them. The bracing below each pin is riveted the arch itself is built of angles and plates with riveted con- shows the joint at A where the tie-rods are connected and USPENSION-ROD from the crown of the arch. This construc- principle as that of the WOODEN ARCH shown by Fig. 42.

Express Company's Receiving Station, New York City. rd, October 22, 1904. The roof-trusses in this building are brick walls at the level of the second-story floor and have ed by *I* beams which form a part of the floor-framing of the truss has a span of 74 ft 4 in and a clear rise of 27 ft. They t 5 in apart and have all connections riveted. Since the

ties are very heavy one might be led to classify these trusses with TRUE A
fixed at the ends; but as the condition of FIXED ENDS rarely obtains in pr
it is better to consider this type of structure as an ARCHED TRUSS with a
or possibly as similar to the type shown in Fig. 75.

Table IV. General Dimensions of a Few Three-Hinged Arche

Location	Span ft	Span in	Rise ft	Rise in	Tie
Syracuse University	101	4	...	...	Floor-beams
Lawson Riding-Academy	106	0	56	2¼	No tie
Machinery Hall, Chicago Exp.	121	10	96	3	†Two 1⅞ × 1⅞
22nd Reg. Armory, New York	134	0	32	6	
Coliseum, Chicago (new)	149	9	66	6	†2½ × 2½
Newark, N. J., Armory	163	6	73	5⅝	†Two 2¾ round
Government Bldg., St. Louis Exp.	172	0	69	9½	9-in I beam
Coliseum, St. Louis	178	6	80	0	No tie
Hartford, Conn., Armory	181	0	90	2½	No tie
Frankfort, Germany, Train-Shed	184	0	94 (about)		No tie
69th Reg. Drill-Hall, New York	189	8	103	4½	†Two 1¾-in
5th Reg. Armory, Baltimore, Md.	190	4	88	0‡	Two channels
47th Reg. Armory, Brooklyn, N. Y.	191	4	84	0	†Two 4 × ⅞-in
Coliseum, Chicago (old)	215	0	73	0	
74th Reg. Armory, Buffalo, N. Y.	227	0	94	0	
Coal-Shed, Wende, N. J.	230	0	...	...	†9 × 7/16-in plate
Jersey City Train-Shed	252	8	89	9¾	Two 12-in I be
Philadelphia Train-Shed	259	0	88	3½	
Broad Street Station, Phila.	300	8	100	4	
Manufactures and Liberal Arts Bldg., Chicago Exp.	368	0	206	4	

Location	Distance, center to center*	Referen
Syracuse University	17 ft 11½ in	R. Aug. 11
Lawson Riding-Academy	32 ft 0 in	R. Dec. 31
Machinery Hall, Chicago Exp.	50 ft 8 in	R. Dec. 24
22nd Reg. Armory, New York	11 and 52 ft	N. May 5
Coliseum, Chicago (new)	22½ to 25 ft	N. Sept. 14
Newark, N. J., Armory	31 and 26½ ft	R. May 26
Government Bldg., St. Louis Exp.	35 ft 0 in	N. Sept. 25
Coliseum, St. Louis	36 ft 8 in	N. Aug. 10
Hartford, Conn., Armory	6 and 52⅔ ft	R. Sept. 12
Frankfort, Germany, Train-Shed	33 ft 6 in	R. Mar. 5
69th Reg. Drill-Hall, New York	6⅝ and 38¾ ft	R. June 1
5th Reg. Armory, Baltimore, Md.	....	R. May 14
47th Reg. Armory, Brooklyn, N. Y.	34 ft 4 in	R. Dec. 23
Coliseum, Chicago (old)	46 ft 8 in	N. Nov. 12
74th Reg. Armory, Buffalo, N. Y.	....	R. June 9
Coal-Shed, Wende, N. J.	22 ft 10¼ in	R. Oct. 3
Jersey City Train-Shed	14½ and 43½ ft	N. Sept. 25
Philadelphia Train-Shed	....	R. July 16
Broad Street Station, Phila.	....	R. June 10
Manufactures and Liberal Arts Bldg., Chicago Exp.	....	N. Sept. 1

* Center to center of end-supports. † Dimensions in in
‡ To lower chord. N. Engineering News. R. Engineering

R. I. Engineering
3, 1907. The type of
n this building is com-
THREE-HINGED ARCH,
n at each support and
; but the two end-pins
a tie and one end-shoe
rollers and hence the

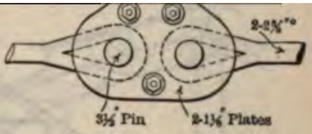


Fig. 71A. Detail at *A*, Fig. 71.

MPLE TRUSS composed of three members, two of which are
elves. The truss-span is 166 ft 8 in and the rise about 61 ft.
iveted and spaced about 26 ft 1 in on centers.

the Pennsylvania Railway Train-Shed, Pittsburgh, Pa.
ord, August 23, 1902. The trusses have three HINGES and a
R-BEARING at one end. The truss-span is 255 ft ⅜ in between
the rise 93 ft between pin-centers and the depth at the center
es are riveted and stand in pairs 9 ft on centers and the pairs
6 in on centers.

inged Arch as employed for supporting the roofs over large
ls, drill-halls, etc., is composed of two CURVED TRUSSES, usually
n and dimensions, resting upon PINS at the supports and con-
over the middle of the span. The supports are assumed to be
and are often connected by a TIE to insure stability and take
l thrust of the arch. While a metal tie between masonry sup-
ake these supports FIXED in position under all or any conditions
r all practical purposes they may be so considered; and these
RUCTURES which have ties, provided there is no arrangement
d-movement due to roller-bearings, etc., may be classified with
ports must resist all horizontal as well as all vertical forces.
s are usually placed below the floor-level so that the tie-rods,
be concealed by the floor or even made a part of its framing.

riveted connections and frequently with a solid-plate web at the bottom. T determining of the stresses and detailing of the members and joints requi services of a competent structural engineer; but the illustrations give enable the architect to decide on the general shape of the trusses for the purp of making preliminary drawings and the computations and detail drawing be made later.

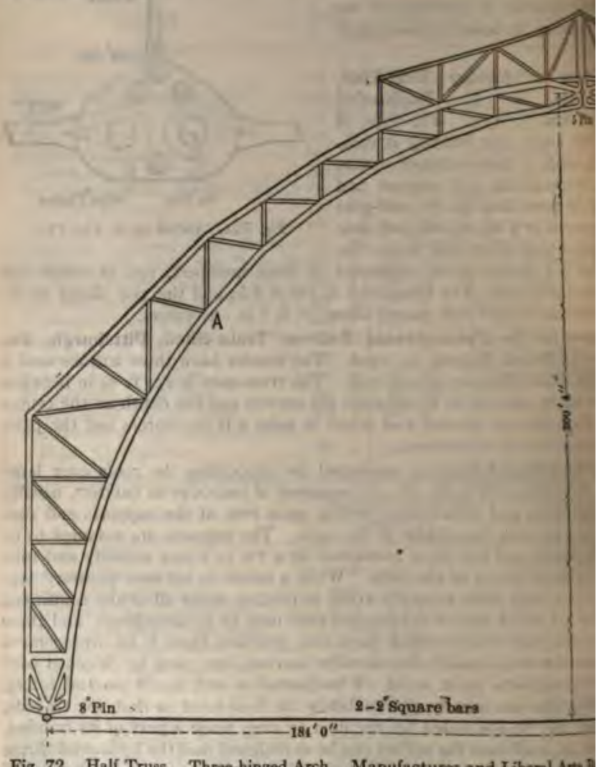


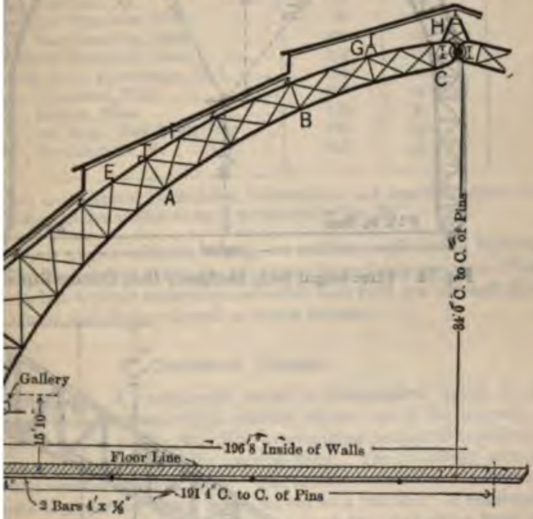
Fig. 72. Half Truss. Three-hinged Arch. Manufactures and Liberal Arts B
Chicago Exposition

Trusses for Railway Station, Frankfort-on-the-Main, German first suggestion for HINGING THE RIBS AT THE CROWN was made by M. a French engineer. The writer believes that the first application of this to roof-trusses, at least on a large scale, was made in the train-sheds of th Railway Station completed in the year 1888 at Frankfort-on-the-Main, G These trusses have a span of about 184 ft. Engineering Record of Se 12, 1891, and March 5, 1892.

Trusses for Machinery Hall, Paris Exposition. The large Machinery Hall of the Paris Exposition of 1899 was supported span being 368 ft and exceeding anything hitherto attempted in Since then trusses of this kind have been frequently used for xhibition-halls, train-sheds, armories, and similar buildings.

r Manufactures and Liberal Arts Building, Chicago Exposi- shows the half-truss of one of the THREE-HINGED ARCHES sup- of of the Manufactures and Liberal Arts Building of the Chicago Engineering News, September 1, 1892.

r Drill-Hall, Brooklyn, N. Y. Fig. 73, in a similar manner, -truss of one of the THREE-HINGED ARCHES over the drill-hall of



3. Half Truss, Three-hinged Arch, Drill-hall, Brooklyn, N. Y.

iment Armory, Brooklyn, N. Y. Engineering Record, December lescription of the arch shown in Fig. 74 is given in the Engineering vember 19 and December 24, 1892. The horizontal thrust due to is small.

d Arches. When there are only two pins, usually at the sup- sses become TWO-HINGED ARCHES. As in the case of three-hinged may be a tie or the supports may be entirely depended upon to izontal thrust.

r Live-Stock Pavilion, Chicago, Ill. In the Engineering News 906, there is a description of the TWO-HINGED ARCHES supporting is building. The arch span is 198 ft, the rise 54 ft and the truss- Each truss has a tie consisting of one 2 9/16-in round rod.

r Railway Station, Cologne, *Germany*. *This station, owned n Railways, has TWO-HINGED ARCHES supporting the roof of the e arch-span is 209 ft 6 in and the rise 79 ft.* There is a brief *the Engineering News, October 6, 1892.* A number of roofs

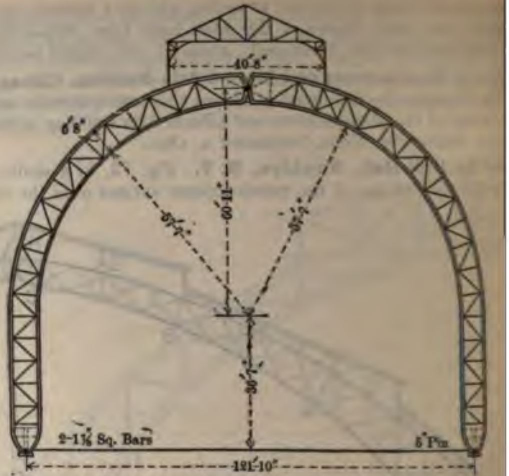


Fig. 74. Three-hinged Arch, Machinery Hall, Chicago Exposition

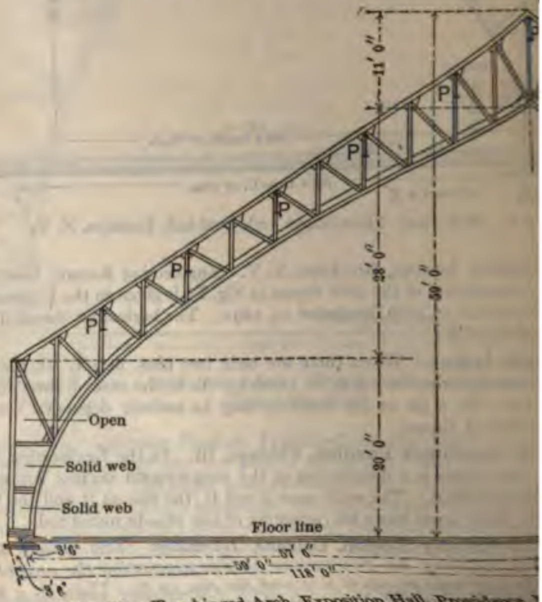


Fig. 75. Two-hinged Arch, Exposition Hall, Providence,

structures similar to that shown in Fig. 75. While such a frame
WO-HINGED ARCH, owing to the lack of freedom at the supports,
, for all practical purposes, be so considered.

t of Buildings with Trusses of the Two-Hinged-Arch Type

Name	Span	Spacing
	ft	ft
, Pawtucket, R. I.	82	24
, Portland, Me.	92	25
Hall, Brockton, Mass.	96	24
, Northampton, Mass.	100	24
ink, Hartford, Conn.	104	25
on Hall, Providence, R. I.	118	24.5
, Cleveland, Ohio	120	23 to 25
, Boston, Mass.	122	30
, 22d Reg., New York City	176	24.5
, Brooklyn, N. Y.	196	35

s are described in Building Construction and Superintendence, Part
ler and are similar to the type shown in Fig. 75.

, or arches without hinges, are seldom employed in buildings.
xamples cited above the structures have the appearance of being
, but a closer inspection indicates that they are not sufficiently
ant their being classed as FIXED ARCHES.

5. Cantilever Trusses

iciples. A CANTILEVER BEAM or CANTILEVER TRUSS is that
er beam or truss which extends beyond one of the supports, as
79 and A in Fig. 80. The overhanging portion B is called the
and the portion C the ANCHOR-SPAN. The cantilever-arm may
d another beam or truss. The term CANTILEVER was originally
a projecting beam which served as a bracket; in engineering
te a beam or girder fixed at one end, by being either built into
nore commonly the case, extended a sufficient distance beyond
m an anchorage. Thus in Fig. 76, which shows a beam resting
B is the cantilever or cantilever-arm and C the anchor-span or
s obvious that if this entire beam were uniformly loaded the
carry the greater part of the total load; and also, that an
, at the end of the cantilever, might cause a negative reaction
t the support D, in which case the reaction at P would exceed
beam, unless the negative reaction at D is considered as an
Although both conditions of loading occur in practice, the
the truss usually requires an anchorage rather than a support
. As applied to roof-construction some such arrangement as
77 is generally required to make this method of support prac-
a wide middle span, with shorter spans or aisles on each side of
ver-arm is usually made from ¼ to ⅓ the middle span and a
RUSS, represented by S, supported by the arms of the canti-
support the rest of the roof. In all such cases, therefore, can-
t be used in pairs, one on each side of the building; and there
ssages outside of the principal span to permit the use of the

outer or anchor-spans. This arrangement is generally foun
armories, exhibition-halls and similar buildings, and is someti
adapted also to other classes of structures. Of course, in a larg
consisting of a single member such as is shown in Fig. 77 c

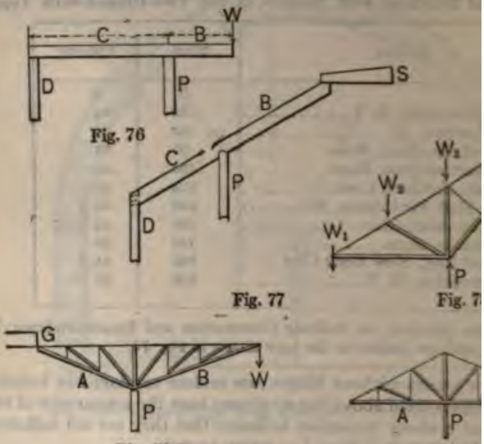


Fig. 76

Fig. 77

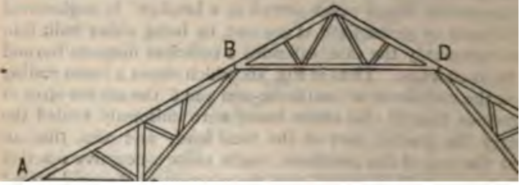
Fig. 7

Fig. 79

Fig.

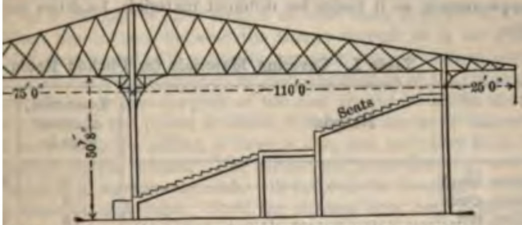
Figs. 76 to 80. Cantilevers and Cantilever Trus

but the principle of construction is the same whether the ca
member or a large truss. Fig. 78 is the diagram of a truss wh
of the beam *CB* in Fig. 77, the single lines representing th
and the double lines the compression-members. Fig. 81 s
arrangement of two of these trusses with the accompanying r



it engineers that the CANTILEVER TYPE OF TRUSS is not an economi-
ot as desirable for spans of 150 ft or more as the THREE-HINGED
s not as readily lend itself to methods of allowing for expansion
n as the THREE-HINGED ARCH, the BOWSTRING TRUSS, or the QUAD-
uss. For certain classes of buildings, however, and especially
ddle span does not exceed 150 ft, it can perhaps be used with
tural effect than is possible with other types, the cost remaining
ne. For roofing platforms, grand-stands, etc., where an outer
desired, it is the only type available.

Grand-Stand, Monmouth Park, N. J. Fig. 82 is a diagram of
NTILEVER TRUSSES supporting the roof of the grand-stand at this



82. Cantilever Truss, Grand-stand, Monmouth Park, N. J.

he details of which were published in Architecture and Building,
1890. This is an instance in which the cantilever was the only
hat could be used and the form adopted is both simple and economi-
be seen from the drawing, the main supporting column extends to
truss, as is usually the case with cantilever trusses, and the truss
ach side of it. The upper and lower chords are made of two angles

CHAPTER XXVII

STRESSES IN ROOF-TRUSSES

By

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1. Roof-Loads. Data, Weights, Materials, Methods

Data for Roof-Trusses. Before the stresses in a roof-truss can mined it is necessary to decide upon the character of the roof-cover method of supporting it between the trusses, the geometrical shape of the trusses and the spacing of the trusses.

Roofing Materials for Pitched Roofs. The materials suitable for pitched roofs are slate, burnt-clay tiles, metal tiles or shingles, wooden corrugated iron, tin with standing seams, standing-seam steel roof various kinds of ready roofing. The least slope to which these mater be laid without danger of leaks, the weight per square foot of roof and t parative cost are indicated in Table I. The cost, however, can only be co as approximate, as it varies for different materials, localities and the wages.

Table I. Covering Materials for Pitched Roofs

Material	Least rise of rafter in 12 in	Comp cost sq
Slates, black	8	$7.00
Slates, green	8	7.00
Slates, red	8	12.00
Burnt-clay tiles, interlocking pattern	7	15.00
Tin shingles, painted	6	8.00
Galvanized-iron tile, painted	6	13.00
Cedar shingles, stained or painted	6	3.80
Corrugated iron, painted	3	4.00
Standing-seam steel roofing, painted	2	4.00
Ready roofing	1	3.50

Roofing Materials for Flat Roofs. Flat roofs or roofs having from ½ to ¾ in to the foot are usually covered with tar and gravel, ready roofing, or tin with lock-and-solder joints. A good tin roof co $8.00 a square, not including the painting. The other kinds vary f to $4.50 a square.

Manner of Supporting the Roof from the Trusses. Wooden ported by wooden trusses, require common or jack-rafters to support th ing or slate, and generally purlins to support the rafters, although in so may be more economical to span the rafters from truss to truss (Fig. 17, XXVI). When slates or burnt-clay tiles are used on steel roofs, they a secured to steel angles, running parallel with the walls and spaced 10½ in apart, as may be necessary to accommodate the size of of anticondensation lining should be provided, as otherwise the e air will condense and fall on the floor or objects below. Flat require rafters and sheathing, or fire-proof filling between the

Trusses. From the above it is seen that the economical spacing depends to a great extent upon the kind of roofing that is used, the span. As a general rule, however, the most economical ut as follows:

TRUSSES under 80-ft span, from 12 to 16 ft on centers.
TRUSSES over 80-ft span, from 16 to 24 ft on centers.
RUSSES under 80-ft span, from 16 to 20 ft on centers.
RUSSES over 80-ft span, from 20 to 40 ft on centers.

G of a number of steel trusses of wide span is given in Chap- Vhen the distance between the trusses exceeds 16 ft for wooden for steel roofs, it is generally necessary to use trussed purlins. d upon the kind of truss to be used, the spacing of the trusses and ruction, a section-drawing of the roof should be made, showing f the truss, the points at which the purlins are to be supported, supporting the ceiling, if there is one, and any other loads that ported by the trusses. The section and truss-drawing, with the weights of roofing-materials, will furnish the necessary data for loads at each joint. Until the stresses have been determined, e members computed, and the joints detailed, an exact drawing nnot, of course, be made; but in order to compute the loads and necessary to know the positions of the joints, and these can be sufficient accuracy before the exact sizes of the members are Chapter XXVI gives sufficient information regarding the various es to enable one to decide upon the height and the number and f the struts and ties; and the sizes of the members can be approx- preliminary drawings.

represents truss 1, of Fig. 2, the roof-area contributory to joint 2 is, in sq $\frac{8+14}{2} \times a$. For truss 2, the area supported by the same joint is $\frac{14+}{}$ or, if we let D represent the length in feet of roof or ceiling supporte joint, the area in square feet supported by joint 2 is $a \times D$, and the

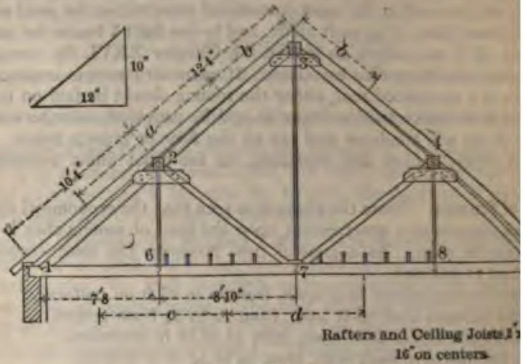


Fig. 1. King-rod Truss

ported by joint 3 is $2\,b \times D$. In the same way, the ceiling-area s joint 6 is $c \times D$, the arrow-heads being half-way between the joints no material difference in the joint-loads whether the common raft ported on purlins or whether they rest on the top chord of the tru the purlins come at or close t and the load is uniformly Thus the width of the ceili utory to joint 7 (Fig. 3) is just the same as in Fig. 1. T ment in Fig. 1 produces c stresses in the tie-beam, wh Fig. 3 does not. When the spaced a uniform distance ap 2, is, of course, equal to t between centers of trusses. trusses are not spaced unif equal to one-half the distan center of the truss on the center of the truss on the righ purlin is more than 12 in fro the roof-area is not symme often the case at hips and joint-load is determined by of the REACTION OF BEAMS, as Chapter IX. Examples showing the computation of joint-loads little farther on.

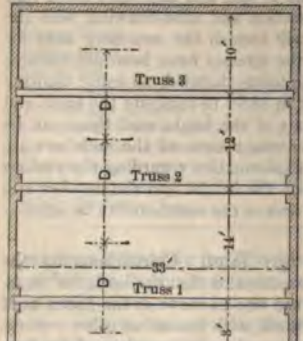
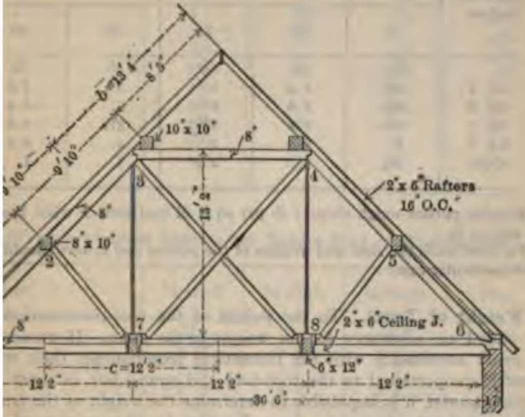


Fig. 2. Plan of Wall and Trusses

Roof-Load per Square Foot. By the term ROOF-LOAD is meant the materials composing the roof, trusses and purlins, an ample

an allowance for wind-pressure. The weight of the materials
EAD LOAD. Snow is generally considered a LIVE LOAD, acting
e pressure due to the wind is always assumed to act normal to,
es to, the surface of the roof; but for trusses of less than 100-ft



Truss. (See, also, Figs. 12, 53 and 54 and Chapter XXVIII, Fig. 1)

ly combined with the DEAD LOAD, WIND-LOAD and SNOW-LOAD and
vertical load.

mputing Dead Loads. The DEAD LOAD of any roof may be
sufficient accuracy from the following data:

ble II. Weights per Square Foot of Roof-Surface

non, 2½ lb; 18 in, 3 lb
hick, 7¼ lb; ¼ in thick, 9.6 lb (the common thickness is 3/16 in for
by 20 in)
lay shingles, 11 to 14 lb
ld style, two parts, 12 lb; new style, one part, 8 lb
ld style, two parts, 19 lb; new style, one part, 8 lb
ntal tiles, 11 lb
, 8 lb
n mortar add 10 lb per sq ft
, sheets, 1½ lb; tiles, 1¾ lb
eets or shingles, including one thickness of felt, 1 lb
n, painted or galvanized, No. 26, 1 lb; No. 24, 1.3 lb; No. 22, 1.6 lb;
b; No. 18, 2.6 lb; and No. 16, 3.3 lb
steel roofing, 1 lb
nd gravel roof, 6 lb
nd gravel roof, 5½ lb
ly roofing (elaterite, rubberoid, asphalt, etc.), from 0.6 to 1 lb
galvanized-iron frame, ¼-in glass, 4½ lb; *5/16-in, 5 lb; 3/8-in, 6 lb*
thick, *3 lb per sq ft for white pine, spruce, or hemlock;* 4 lb for yel-
ine

Table III. Weights of Rafters per Square Foot of Roof-Surface

Size of rafter in inches	Spruce, hemlock, white pine. Spacing in inches, center to center			Hard pine. Spacing in in center to center	
	16	20	24	16	20
	lb	lb	lb	lb	lb
2× 4	1½	1.2	1	2	1.6
2× 6	2¼	1.8	1½	3	2.4
2× 7	2⅝	2.1	1¾	3½	2.8
2× 8	3	2.4	2	4	3.2
2×10	3¾	3	2½	5	4

Wooden purlins weigh about 2 lb per sq ft of roof-surface when the span i 12 and 16 ft.

For steel roofs the sizes and weights of the purlins and rafters should be com each particular case.

Weight of Truss. To the weight of the roof-construction prope be added an allowance for the weight of the truss. If trusses could in exact accordance with the theoretical requirements their weight w directly proportional to the roof-load and span; but as there is alwa extra material, it is impossible to determine the weight of the truss exac it is completely designed. Several tables for the weights of wooden and formulas for steel trusses have been published, but hardly any two are alike. The following are some of the formulas in use:

For Wooden Trusses

$W = 0.04\,L + 0.000167\,L^2$	N. C. Ricker, for trusses lik and 5, Chapter XXVI.
$W = 0.05 + 0.075\,L$	H. S. Jacoby.
$W = 0.75 + 0.075\,L$	Mansfield Merriman.

For Steel Trusses

$W = 0.6 + 0.06\,L$, for heavy loads $W = 0.4 + 0.04\,L$, for light loads	C. E. Fowler, for Fink trus
$W = P/45\left(1 + \frac{1}{5}\sqrt{A}\right)$	M. S. Ketchum, for steel m ing trusses.
$W = 0.05\,L + 12/A$	H. G. Tyrrell.

In the above formulas, W = weight of truss in pounds per square foo zontal projection of the roof supported, L = span in feet, A = distance trusses, and P = capacity of truss in pounds per square foot of horizo jection.

Tables IV and V, compiled from a comparison of other tables and and from the weights of actual trusses, are sufficiently accurate for the of determining stresses. The weights given are probably slightly in the actual weights of average trusses, as it is preferable to have the erro on the safe side. It should be noted that the weights are for each sq of roof-surface, and not for the horizontal area. Table VI gives t weights of a number of large steel roofs.

eights of Wooden Trusses per Square Foot of Roof-Surface*

an	½ pitch	⅓ pitch	¼ pitch	Flat
	lb	lb	lb	lb
	3	3½	3¾	4
	3¼	3¾	4	4½
	3½	4	4½	4¾
	3¾	4½	4¾	5¼
	4¼	5	5½	6
	5	6	6½	7
	5¾	6¾	7	8
	6½	7½	8	9
	7	8½	9	10

Weights of Steel Trusses per Square Foot of Roof-Surface

an	½ pitch	⅓ pitch	¼ pitch	Flat
	lb	lb	lb	lb
	5.25	6.3	6.8	7.6
	5.75	6.6	7.2	8.0
	6.75	8.0	8.6	9.6
	7.25	8.5	9.2	10.2
	7.75	9.0	9.7	10.8
	8.5	10.0	10.8	12.0
	9.5	11.0	12.0	13.2
	10.0	11.6	12.6	14.0

ghts and Spacing of Some Steel Roofs of Wide Span, Including s, Purlins and Braces, but not Roof-Covering or Rafters

uilding	Type of truss	Span ft	Spacing, center to center of trusses, ft	Weight per sq ft sloping surface, lb	Weight of one truss, tons
ucket, R. I.	Fig. 75†	82	24	8.7	6.7
and, Me.	"	92	25	9.7	9
, Brockton,	"	96	24	8.6	10
orthampton,	"	100	24	8.0	8.5
Hartford,	"	104	25	11.8	11.5
idence, R. I.	"	118.	24½	9.5	12.5
io, Armory	"	120	23-25	....	....
n, Mass.	"	122	30	12.4	21
egt., N. Y.	"	176	24½	....	....
clyn, N. Y.	"	196	35	....	....

---

* For scissors trusses, increase one-third.

† Chapter XXVI.

pitch.

Table VII. Allowance for Snow in Pounds per Square Foot

Location	Pitch of ro		
	½	⅓	¼
	* †	* †	* †
Southern states and Pacific slope......	0– 0	0– 5	0– 5
Central states..........................	0– 5	7–10	15–20
Rocky Mountain states.................	0–10	10–15	20–25
New England states....................	0–10	10–15	20–25
Northwest states.......................	0–12	12–18	25–30

Columns headed by an asterisk (*) are for slate, tile, or metal; dagger (†) are for shingles.

Wind-Pressure.* For roofs having a pitch of 5 in or m allowance must be made for wind-pressure. For trusses of th or QUEEN TYPES, the usual practice is to include the wind- vertical loads, and to make a single allowance for both wind a a gale snow is not likely to stay on a steep roof. When th added to the vertical loads, the allowance for wind and snow not be less than indicated in Table VIII.

Table VIII. Allowance for Wind and Snow Combined i Square Foot of Roof-Surface

Location	Pitch of roof			
	60°	45°	⅓	¼

ce maximum stresses, or call for COUNTERBRACING, as is the case AL TRUSSES, and trusses with CURVED CHORDS, the stresses for all dings should be found separately and each member of the truss r the maximum stress to which it may be subject under any ation of loads. For determining the stresses due to wind-pres-orce of the wind is usually assumed to act in a direction normal, angles, to the slope of the roof. This force is commonly based on d, producing a pressure of 30 lb against a vertical surface. This a wind-velocity of nearly 100 miles per hour. According to la,

$$P = 0.0032\ V^2$$

pressure in lb per sq ft against a surface normal to the direction V = the velocity in miles per hour. For $P = 30$ lb, $V = 96.3$ rmal pressure per square foot of roof-surface corresponding to nd 30 lb per sq ft against a vertical surface is given in Table IX.

Wind-Loads in Pounds per Square Foot of Roof-Surface

Inclination of roof	Normal pressure P_n, pounds per square foot	
	$P = 30$ lb	$P = 20$ lb
	5.1	3.4
	10.1	6.7
	14.6	9.7
h	19.8	13.1
h	22.4	14.0
	24.0	16.0
h	25.5	17.0
	26.7	17.8
	28.3	18.9
	30.0	20.0

n Table IX are based on Duchemin's formula,

$$P_n = P\frac{2\sin\theta}{1+\sin^2\theta}$$

he pressure per square foot on a vertical surface, P_n the normal ressure and θ the angle of inclination of the roof with the hori-ind not only produces a pressure upon the windward side of the ion upon the leeward side; therefore all roof-covering should be d, all joints in the trusses so constructed that they will resist mpression, and the trusses themselves securely anchored to the

n Loading for which Stresses should be Found. To deter-mum stresses under any possible condition of loading, stresses l for the following cases:

s due to permanent DEAD LOADS,
covering only one side of roof,
covering *entire roof*,
n side of truss nearer the expansion-end,
side of truss nearer the fixed end.

following conditions: First, full dead and live load on both
roof-garden, and wind-pressure due to a velocity of 45 miles
full dead load, snow-load, and gallery live load, wind-press
load on roof-garden floor; third, full dead load and 50 lb wind
full dead load and wind-pressure at 45 miles an hour, and
gallery and roof-garden on one side only. Snow-loads thro
at one-third of the dead load. Examples showing manner
stresses due to different conditions of loading are given on pag

2. Examples of the Computation of Roof-L

King-Rod Truss. **Example 1.** The first example cons
truss shown in Fig. 1, page 1048, which it is assumed represen
It is assumed that the timber is to be common white pine a
to be covered with 3/16-in slate of medium size on 7/8-in shea
is to consist of lath and plaster. The dead load of roof a
foot of roof-surface is made up as follows:

For slate. . . . .
For sheathing. . . . .
For rafters. . . . .
For purlins. . . . .
For truss. . . . .
Total. . . . .

For wind and snow-load combined there should be allowe
pitch being about 40°), which makes a total roof-load of
fractions, however, the load is assumed to be 48 lb per sq f
to truss 1, Fig. 2, is 14 ft and to truss 3, 12 ft, the length o
the truss is 13 ft. The roof-area supported by the purlins a

sts and 10 lb for the lath and plaster; but where there is a large le to be used for storage it is well to make a small allowance, sq ft, for any extra attic-load. Therefore, 18 lb per sq ft is e weight of the ceiling, which makes the weight at joints 6 and y 18 lb per sq ft, or 1 930 lb; and the weight at joint 7, 114⅝ sq sq ft, or 2 067 lb. As soon as computed, the roof and ceiling- marked on a truss-diagram, as in Fig. 10. The roof and ceiling- are transmitted directly to the wall and need not be taken into rmining the stresses in the truss.

ss. Example 2. It is required to compute the joint-loads for n in Fig. 3, page 1049. All timber is to be of spruce and the roof d with shingles on 1-in sheathing. The ceiling is to be of lath he dead load is:

	lb per sq ft
shingles	2½
sheathing	3
rafters	2¼
purlins	2
truss	3
dead load per sq ft	12¾
for wind and snow	30
roof-load in pounds per square foot	42¾

of the ceiling it is well, for a truss of this kind, to allow at least It will be assumed that the trusses are to be spaced uniformly rs. Then the roof-area supported at joint 2 is 9⅚ by 15 ft, or d the load at this joint is 6 306 lb. The purlin at joint 3 supports point midway to joint 2, to the ridge, or $b = 4$ ft 11 in + 8 ft 5 in, The roof-area supported at this joint is 13⅓ by 15 ft, or 200 sq d is 8 550 lb. The loads at joints 4 and 5 are equal respectively nd 2. For the ceiling-loads at joints 7 and 8 there is an area to be al to 12⅙ by 15 ft, or 182½ sq ft, which, multiplied by 20, gives

russ. Example 3. For this example the church-roof shown in 4 is considered. In this roof the trusses take the place of the iling-beams, the sheathing spanning from truss to truss and the iling being nailed to 1¼ by 2½-in furring strips, spaced 12 or 16 in ssuming that the parts of the trusses have the dimensions indi- gure, and that the wood is white pine, the actual weight of one 1 200 lb. The roof-area supported by one truss is 170 sq ft, and ght of the trusses is about 7 lb per sq ft of roof-surface. This than twice that given in Table IV, owing principally to the close trusses and also to the small dimensions of their members. The heathing and shingles is about 5½ lb and 30 lb is allowed for wind- roof is too steep for snow to lodge on it. This gives a total roof- per sq ft of sloping surface. For the weight of the ceiling 12 lb ple, as no load other than its own weight is likely to come upon it. supported at joint 2 is 10⅚ by 2½ ft, or 27 sq ft. The area sup- ts 4 and 5 is equal to 12⅖ by 2½ ft, or 31 sq ft for each. The pported at joint 3 is 14⅕ by 2½ ft, or 35½ sq ft. Multiplying y the corresponding loads per square foot, there results 1 148 lb for the load at joint 2, 1 318 lb for each load at joints 4 and 5, and the load at joint 3.

Truss over Car-Barn. Example 4. In this example the roof is gated iron, supported by a steel truss of the shape shown in Fig. 55, XXVI. This truss supports nothing but the corrugated iron, the the pressure due to wind and snow, the use of the building not requ suspending of any load from the trusses. In figuring the dead loads a roof, the sizes of the purlins and the gauge of the iron should first be

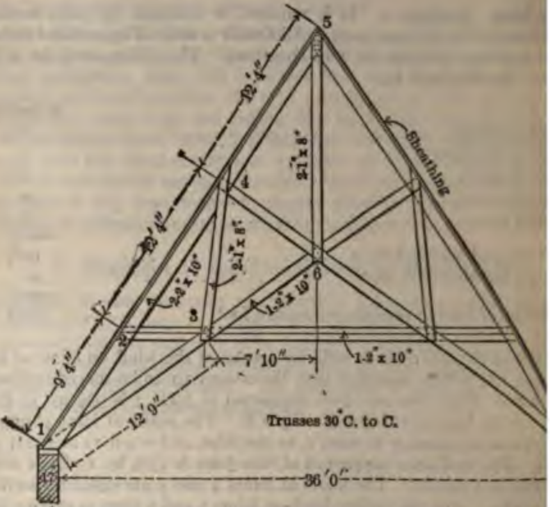


Fig. 4. Scissors Truss. (See, also, Fig. 24 and Chapter XXVIII, F

fixed, so that the weight per square foot of roof may be accurately In this instance the purlins are 5-in **I** beams spaced 4 ft 9 in on c weighing 10 lb per linear foot. The weight of the purlins per squ roof is therefore equal to 10 lb divided by 4¾, or 2.1 lb. For a span the corrugated iron should be No. 18 gauge (see Corrugated Iron, Pa 1515) weighing 2 lb per sq ft. For the weight of the truss and bracin taken is that given in Table V for a span of 100 ft and ¼-pitch, 10.8 gives a total dead load of 14.9 lb per sq ft of sloping surface.

For wind and snow we should allow 22 lb per sq ft if the building in the Central states, making the total roof-load 36.9 lb per sq ft. generally recommended, however, that no roof should be designed less, all told, than 40 lb per sq ft; the joint-loads, therefore, should b on that basis. The only loaded joints in this truss are those under The trusses are spaced 19 ft 2¼ in and the purlins 4 ft 9 in on center area supported at each upper joint being 91 sq ft. The joint-loads should be figured at 3 640 lb. Even for the locality in which it was

---

* The actual weight of this truss and bracing was 4 lb per sq ft which is remarkably small.

t roof; and it would hardly be considered safe for states further
.

Flat Roof. **Example 5.** This truss is for a flat roof (Fig. 5). of spruce and there is a five-ply gravel roof and a plastered ceiling. load we have,

	lb per sq ft
f roofing..................................	6
f sheathing................................	3
f rafters..................................	2¼
f purlins..................................	2
f truss, about.............................	4¼
l dead load in pounds per square foot	17½

is required for wind-pressure, but the snow-load is a large per- total load in any of the Northern states, as indicated in Table VII.

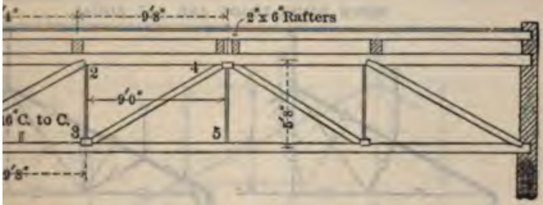


Fig. 5. Howe Truss

t the building is located in one of the Central states, 30 lb per sq llowed for snow, making the total roof-load 47½ lb. The plaster ne ceiling-joists weigh about 12¼ lb and as the roof-space is not sed for storage, 13 lb per sq ft is a sufficient allowance for the ceil- ng that the trusses are to be uniformly spaced, 14 ft on centers, supported at joint 2 is 9½ by 14 ft, or 133 sq ft, and the area sup- nt 4, 9⅔ by 14 ft, or 135⅓ sq ft. The ceiling-area supported at by 14 ft, or 130⅔ sq ft and at joint 5, 9 by 14 ft, or 126 sq ft. Mul- of these areas by the corresponding load per square foot, we have ne load at joint 2, 6 428 lb at joint 4, 1 699 lb at joint 3, and 1 638 lb n practice it is hardly worth while to compute the stresses closer so that the loads may as well be put down at an even 50 or 100 lb ads obtained by computation. When the roof is supported by are often some joints of the truss which have no load. Thus for wn in Fig. 16, Chapter XXVI, there are no loads on joints 2, 6 and -area supported at joint 4 (Fig. 16) is equal to one-half the distance l by the distance halfway to the truss on each side. If the lower ts ceiling-joists, there is a load at each of the joints 3, 5, 7, 9, etc. AMS can be drawn for any arrangement of loads, the important compute the loads exactly as they are placed on the truss.

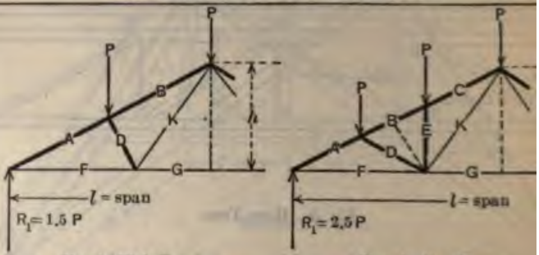
examples illustrate fairly well the *method of computing the loads* types *of trusses. Other special cases of loading should be com- ame principles.*

3. Determination of Stresses by Computation

Stresses. To determine the stresses, a DIAGRAM OF THE TRUSS, con of single lines representing the central axial or median lines of the truss-me should first be carefully drawn to a scale and the loads at the differen indicated by arrows and numbers as in Figs. 10 and 12. If the center the members, as they are actually placed, do not intersect at commo they must be made to do so in the diagram, as the stresses can be COMPUT on the assumption that the center lines of all members meeting at intersect at a common point. In wooden trusses it is not always practi place the members so that their center lines meet in a common point joint; but this condition should obtain as nearly as practicable, and trusses the joint-connections should be made so that the lines passing the centers of gravity of the cross-sections of the members meeting at intersect in the same point.

Table X. Coefficients for Determining the Stresses in Simple Fink Fan Trusses

WHEN PANEL-LOADS ARE ALL EQUAL



Simple Fink Truss Simple Fan Truss

To find the stress in any member, multiply its factor by the panel-load

SIMPLE FINK TRUSS

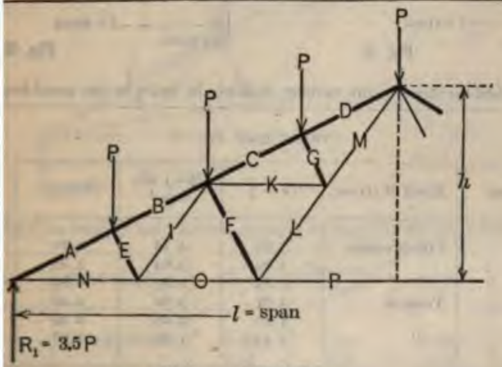
Member	Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$
A............	Compression	2.70	3.00	3.35
B............	"	2.15	2.50	2.91
D............	"	0.83	0.87	0.89
F............	Tension	2.25	2.60	3.00
G............	"	1.50	1.73	2.00
K............	"	0.75	0.87	1.00
		SIMPLE FAN TRUSS		
A............	Compression	4.51	5.00	5.59
B............	"	3.54	4.00	4.55
C............	"	3.40	4.00	4.70
D............	"	0.93	1.00	1.08
E............	"	0.93	1.00	1.08
F............	Tension	3.75	4.33	5.00
G............	"	2.25	2.60	3.00
K............	"	1.50	1.73	2.00

ation of Stresses. As a general rule, the stresses in a roof-truss can ned much more readily by the GRAPHIC METHOD than by MATHEMATI- TATIONS and with as close a degree of accuracy as is necessary. There forms of trusses, however, for which the stresses can be more easily l by COMPUTATION. Such trusses must be symmetrical in shape nt-loads all alike, as is quite frequently the case with simple steel roofs ceiling-load.

X to XIII give constants by which the stresses in Fink and fan y be readily COMPUTED simply by multiplying the constant by the oint-load. These tables apply, however, only when the rafter is the struts into equal spaces, giving equal panel-loads. For any itions the stresses should be determined by the GRAPHIC METHOD.

XI. Coefficients for Determining the Stresses in an Eight-Panel Fink Truss

WHEN PANEL-LOADS ARE ALL EQUAL



Eight-panel Fink Truss

nd the stress in any member, multiply its factor by the panel-load, *P*

	Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
....	Compression	6.31	7.00	7.83	9.42
....	"	5.76	6.50	7.38	9.05
....	"	5.20	6.00	6.93	8.68
....	"	4.65	5.50	6.48	8.31
....	"	0.83	0.87	0.89	0.93
....	"	1.66	1.73	1.79	1.86
....	"	0.83	0.87	0.89	0.93
....	Tension	0.75	0.87	1.00	1.25
....	"	0.75	0.87	1.00	1.25
....	"	1.50	1.73	2.00	2.50
....	"	2.25	2.60	3.00	3.75
....	"	*5.25*	*6.06*	*7.00*	*8.75*
....	"	*4.50*	*5.19*	*6.00*	*7.50*
....	"	*3.00*	*3.46*	*4.00*	*5.00*

Table XII. Coefficients for Determining the Stresses in Cambered F and Fan Trusses

WHEN PANEL-LOADS ARE ALL EQUAL AND THE CAMBER EQUALS ON SIXTH THE RISE

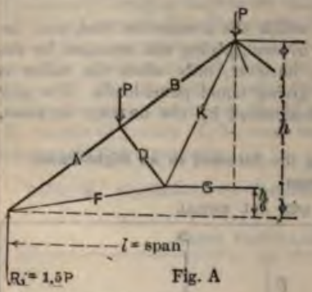


Fig. A

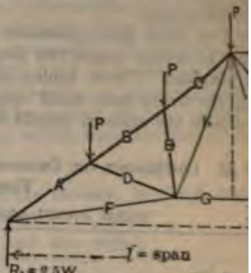


Fig. B

To find the stress in any member, multiply its factor by the panel-load,

TRUSS LIKE FIG. A

Member	Kind of stress	$l/h = 3$	$l/h = 3.464$ $= 30°$	$l/h = 4$
A............	Compression	3.64	4.13	4.70
B............	"	3.09	3.63	4.25
D............	"	0.83	0.87	0.89
F............	Tension	3.07	3.62	4.24
G............	"	1.80	2.08	2.40
K............	"	1.43	1.69	1.98

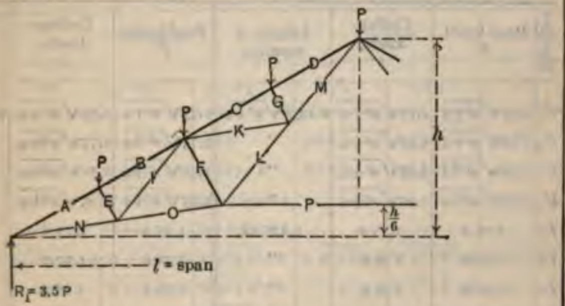
TRUSS LIKE FIG. B

Member	Kind of stress	$l/h = 3$	$l/h = 3.464$ $= 30°$	$l/h = 4$
A............	Compression	6.09	6.83	7.83
B............	"	4.89	5.63	6.48
C............	"	4.96	5.88	6.93
D............	"	1.04	1.15	1.26
E............	"	1.04	1.15	1.26
F............	Tension	5.12	6.03	7.07
G............	"	2.70	3.12	3.60
K............	"	2.66	3.13	3.67

Table XIV gives coefficients which are general for any span and d eight-panel roof-trusses with the Howe and Pratt types of bracing. Ta and XVI give formulas for COMPUTING the stresses in symmetrical Ho Pratt trusses which are symmetrically loaded. The coefficients are gi trusses having an odd number of panels. For the Howe truss with a number of panels the coefficients for the center load on the top chord a divided by two. For the center load on the bottom chord the coeffi also divided by two, except that for the center vertical, which re 'ratt truss with an even number of panels the coefficients are divided or the center loads for all pieces, except that for the center vertical on the top chord, the coefficient remains unity. For the young architect er these tables will be found useful in furnishing a check upon stresses ed by GRAPHIC METHODS.

XIII. Coefficients for Determining the Stresses in an Eight-Panel Cambered Fink Truss

'ANEL-LOADS ARE ALL EQUAL AND CAMBER EQUALS ONE-SIXTH THE TOTAL RISE



find the stress in any member, multiply its factor by the panel-load, *P*

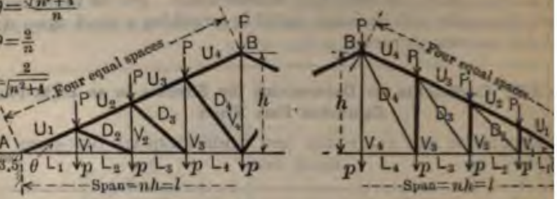
er	Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
......	Compression	8.49	9.63	10.96	13.49
......	"	7.94	9.13	10.51	13.11
......	"	7.39	8.63	10.06	12.74
......	"	6.83	8.13	9.61	12.37
......	"	0.83	0.87	0.89	0.93
......	"	1.66	1.73	1.79	1.86
......	"	0.83	0.87	0.89	0.93
......	Tension	1.02	1.21	1.41	1.80
......	"	1.02	1.21	1.41	1.80
......	"	2.87	3.37	3.96	5.04
......	"	3.89	4.58	5.37	6.85
......	"	7.17	8.44	9.90	12.61
......	"	6.15	7.23	8.48	10.81
......	"	3.60	4.16	4.80	6.00

Table XIV. Coefficients for Eight-Panel Roof-Trusses

$\text{Sec}\ \theta = \frac{\sqrt{n^2+4}}{n}$

$\text{Tan}\ \theta = \frac{2}{n}$

$\text{Sin}\ \theta = \frac{2}{\sqrt{n^2+4}}$

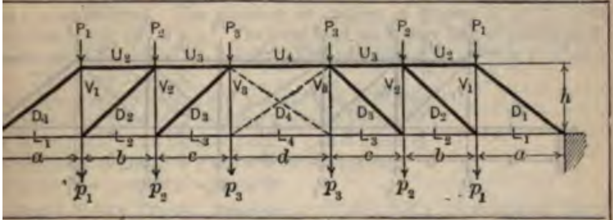


Member	Roof-loads, P	Ceiling-loads, p	Length of member	Roof-loads, P	Ceiling-loads, p	Len me
U_1	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	$0.125h\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	0.125h
U_2	$1.50\sqrt{n^2+4}$	$1.50\sqrt{n^2+4}$	"	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	
U_3	$1.25\sqrt{n^2+4}$	$1.25\sqrt{n^2+4}$	"	$1.50\sqrt{n^2+4}$	$1.50\sqrt{n^2+4}$	
U_4	$1.00\sqrt{n^2+4}$	$1.00\sqrt{n^2+4}$	"	$1.25\sqrt{n^2+4}$	$1.25\sqrt{n^2+4}$	
L_1	$1.75\ n$	$1.75\ n$	$0.125\ nh$	$1.75\ n$	$1.75\ n$	0.1
L_2	$1.75\ n$	$1.75\ n$	"	$1.50\ n$	$1.50\ n$	
L_3	$1.50\ n$	$1.50\ n$	"	$1.25\ n$	$1.25\ n$	
L_4	$1.25\ n$	$1.25\ n$	"	$1.00\ n$	$1.00\ n$	
V_1	0	1.0	$0.25\ h$	1.0	0	0.2
V_2	0.5	1.5	$0.50\ h$	1.5	0.5	0.5
V_3	1.0	2.0	$0.75\ h$	2.0	1.0	0.7
V_4	3.0	4.0	$1.00\ h$	0	1.0	1.0
D_2	$0.25\sqrt{n^2+4}$	$0.25\sqrt{n^2+4}$	$0.125h\sqrt{n^2+4}$	$0.25\sqrt{n^2+16}$	$0.25\sqrt{n^2+16}$	0.125h
D_3	$0.25\sqrt{n^2+16}$	$0.25\sqrt{n^2+16}$	$0.125h\sqrt{n^2+16}$	$0.25\sqrt{n^2+36}$	$0.25\sqrt{n^2+36}$	0.125h
D_4	$0.25\sqrt{n^2+36}$	$0.25\sqrt{n^2+36}$	$0.125h\sqrt{n^2+36}$	$0.25\sqrt{n^2+64}$	$0.25\sqrt{n^2+64}$	0.125h

Stress = coefficient × P or p.

For a half-truss supported at A and B, reduce all top chord coefficients by and all bottom-chord coefficients by n. The coefficients for the web-memb remain unchanged.

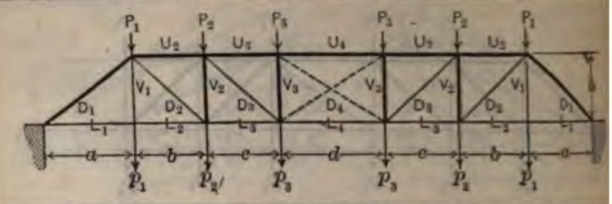
e XV. Coefficients for Howe Trusses which are Symmetrical About the Center of the Span and Symmetrically Loaded



ber	7 panels			5 panels		3 panels
	P_1	P_2	P_3	P_1	P_2	P_1
U_2	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$
U_3	$a \div h$	$(a+b) \div h$	$(a+b) \div h$	$a \div h$	$(a+b) \div h$	....
U_4	$a \div h$	$(a+b) \div h$	$(a+b+c) \div h$	....	....	....
....	$a \div h$	$(a+b) \div h$	$(a+b+c) \div h$	....	....	....
....	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$
....	0	$\sqrt{b^2+h^2} \div h$	$\sqrt{b^2+h^2} \div h$	0	$\sqrt{b^2+h^2} \div h$	....
....	0	0	$\sqrt{c^2+h^2} \div h$	0	0	....
....	0	0	0	....	....	....
....	0	1.0	1.0	0	1.0	0
....	0	0	1.0	0	0	....
....	0	0	0	....	....	....
	p_1	p_2	p_3	p_1	p_2	p_1
...	1.0	1.0	1.0	1.0	1.0	1.0
...	0	1.0	1.0	0	1.0	....
...	0	0	1.0	....	....	....

ads p_1, p_2, etc., the coefficients for the chords and diagonals are the same as
the loads P_1, P_2, etc. The coefficients for the verticals for loads p_1, p_2, etc.
n in the supplementary table below the general table. Tension is indicated
russ diagram by light lines.

Table XVI. Coefficients for Pratt Trusses which are Symmetrical About the Center of the Span and Symmetrically Loaded



Member	7 panels.			5 panels		3 pan
	P_1	P_2	P_3	P_1	P_2	P_1
L_1 and L_2	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$
L_3 and U_2	$a \div h$	$(a+b) \div h$	$(a+b) \div h$	$a \div h$	$(a+b) \div h$	....
L_4 and U_3	$a \div h$	$(a+b) \div h$	$(a+b+c) \div h$	....	....	....
$U_4 = L_4$...	....	....	....	....	....	....
D_1.......	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2}$
D_2.......	0	$\sqrt{b^2+h^2} \div h$	$\sqrt{b^2+h^2} \div h$	0	$\sqrt{b^2+h^2} \div h$	....
D_3.......	0	0	$\sqrt{c^2+h^2} \div h$	0	0	....
D_4.......	0	0	0	....	....	....
V_1.......	0	0	0	0	0	0
V_2.......	0	1.0	1.0	0	1.0	....
V_3.......	0	0	1.0	....	....	....
	p_1	p_2	p_3	p_1	p_2	p_1
V_1.......	1.0	0	0	1.0	0	1.0
V_2.......	0	0	1.0	0	0	....
V_3.......	0	0	0	....	....	....

For loads p_1, p_2, etc., the coefficients for the chords and diagonals are the sa given for the loads P_1, P_2, etc. The coefficients for the verticals for loads p_1, p are given in the supplementary table below the general table. Tension is indi in the truss-diagram by light lines.

mples Showing Use of Tables in Stress-Computations

an Truss. **Example 1.** In this example a simple fan truss of
s considered. The distance on centers of trusses is 12 ft. The
ss is 9 ft, or $l/h = 4$. The total load per square foot of roof is 40 lb.
of rafter is 20 ft, nearly. The panel-load, $P = \frac{20}{3} \times 12 \times 40 =$
hen from Table X,
ower end of rafter $A = 3\,200 \times 5.59 = 17\,888$ lb
nds of main tie $F = 3\,200 \times 5.00 = 16\,000$ lb
enter of main tie $G = 3\,200 \times 3.00 = 9\,600$ lb
races D and $E = 3\,200 \times 1.08 = 3\,456$ lb
ie $K = 3\,200 \times 2 = 6\,400$ lb

el Howe Truss. **Example 2.** (Table XV.) A five-panel Howe
dered, for which $h = 6$ ft, $a = 9$ ft, $b = 10$ ft and $c = 12$ ft. Let the
aced 10 ft on centers, the roof-load be 40 lb per sq ft and the ceiling-
er sq ft. The panel-loads become:

$$\left.\begin{array}{l}(9+10)\,(10\times 40) = 3\,800\text{ lb}\\ (9+10)\,(10\times 15) = 1\,400\text{ lb}\end{array}\right\} = 5\,200\text{ lb}$$

$$\left.\begin{array}{l}(10+12)\,(10\times 40) = 4\,400\text{ lb}\\ (10+12)\,(10\times 15) = 1\,700\text{ lb}\end{array}\right\} = 6\,100\text{ lb}$$

$U_2 = \frac{9}{6} \times 5\,200 + \frac{9}{6} \times 6\,100 = 17\,000$ lb
$U_3 = \frac{9}{6} \times 5\,200 + \frac{19}{6} \times 6\,100 = 27\,100$ lb
$82/6\,(5\,200 + 6\,100) = 20\,400$ lb
$66/6 \times 6\,100 = 11\,900$ lb
$00 + 1\,400 + 1\,700 = 7\,500$ lb
00 lb

results all values between 50 and 100 have been considered 100.

nination of Stresses in Roof-Trusses by Graphic Methods

hic Method is the simplest and in most cases the quickest method
ng the stresses in a roof-truss; and it has, besides, the additional
f being applicable to any true truss-form or any arrangement of
e is also less chance of making a mistake in the GRAPHIC METHOD
method of NUMERICAL COMPUTATION, as an error in the graphical
ost always becomes manifest. When the principles are under-
S-DIAGRAMS can be very quickly drawn, without the aid of books
or the forms of trusses in common use, the method of drawing the
ms is quite simple; and a careful study of the following examples,
d by a little practice in drawing the diagrams, should enable any
aughtsman, or builder to understand the principles involved in the
NALYSIS OF ROOF-TRUSSES.

s Upon Which the Graphic Method is Based. To thoroughly
his method, a knowledge of the COMPOSITION AND RESOLUTION OF
explained in Chapter VI, is essential; and before studying this
student should read carefully pages 288 and 289. The theorems
explained on these pages form the basis of GRAPHIC STATICS.
PHIC METHOD all forces, including the loads, are represented
lines, and the directions of the forces must be constantly kept in
n it is of assistance to indicate the direction of a force by an arrow-
lained on page 289. The direction in which a force acts with refer-
dy indicates, also, whether it is a *PUSHING or a PULLING* force, or
member on which the force or in which the stress acts is in COMPRES-
ON. This is more fully explained in the following pages, and also
ith several of the stress-diagrams.

each side of the middle of the span, the supporting forces are ec
equal to one-half the total load on the truss.

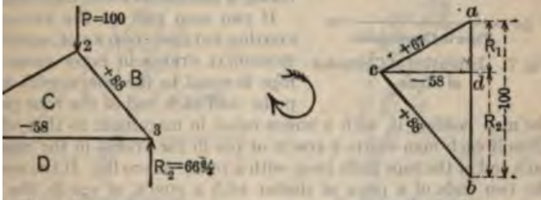
Unsymmetrical Loads. When the loads are not symme
middle, either in regard to point of application or to magnitude
forces are unequal and in most cases must be determined befor
gram can be drawn. The supporting forces for unsymmetrical
may be computed by the method of the MOMENTS OF FORC
pages 322 to 324.

Stress-Diagrams for Vertical Loads. Before the stress-di
can be drawn, it is necessary to make a skeleton drawing of the tr
the central or median lines of the members as explained on p
diagram, called the TRUSS-DIAGRAM, should be drawn on th
paper as the STRESS-DIAGRAM, for convenience in drawing
truss-diagram should also have all of the loads which come on th
by arrows and figures, as in the following examples.

Supporting Forces. The SUPPORTING FORCES, also, should
the TRUSS-DIAGRAM as in Fig. 10. These forces are determi
on pages 322 to 324.

Lettering the Truss-Diagram. After the truss-diagram is
venient to letter it according to the method known as Bow's N
allows a ready comparison of the TRUSS-DIAGRAM and the
and also enables the student to readily draw the stress-diagram
ately determine the CHARACTER as well as the MAGNITUDE of th
essential principle of this method is the LETTERING of each sp
of every external force and of every member of the truss, so th
diagram a truss-member or external force is denoted by the
side of it. When the stress-diagram is drawn, it will be foun
letters come at the ends of the lines representing the externa
stresses in the truss-members.

laid off to scale, usually in order, the frame or truss upon which ot be moved either vertically or horizontally by the forces. The N should always be drawn and closed before any attempt is made the stresses in the members of the truss. The stresses in the e truss will now be found, beginning with those meeting at joint 1. CD meet at this joint. The stresses in these two pieces and R_1 are M and, consequently, if laid off in order will form a CLOSED FIGURE Chapter VI. In diagram 2, *da* represents R_1 in MAGNITUDE and rom *a* draw a line parallel to *AC* and from *d* a line parallel to *CD*



. Truss-diagram

2. Stress-diagram

Fig. 6. Triangular Frame

em until they intersect at *c*. *ac* is the stress in *AC*, and *cd* that and *da*, or R_1, are in EQUILIBRIUM since they form a CLOSED FIG- the forces in order, *da*, or R_1, is known to act towards the joint. *ac* is also towards the joint and hence the stress is of the same he force R_1 and the piece *AC* is in COMPRESSION. *AC* pushes nt as R_1 does. Continuing around the stress-polygon *dac*, in the , *cd* acts away from the joint and the stress in *CD* is opposite in he force R_1, or *CD* is in TENSION. *CD* pulls away from joint 1. stresses in the pieces *BC* and *CA* and the force *AB* are in EQUI- sides of the STRESS-POLYGON are *ab*, *bc* and *ca* (diagram 2). The represents the load of 100 lb acts down and towards the joint, act towards this joint, showing that the stresses in *BC* and *CA* e character as the force *AB*, or that the pieces push against the each is in COMPRESSION. At joint 3, the two pieces meeting are The STRESS-POLYGON is *bdc*. Here *bd* acts towards the joint, *dc* joint, and *cb* towards the joint. As found before, the stress in and that in *CB*, COMPRESSION. Diagram 2 is made up of three NS, one for each of the joints shown in diagram 1. Each of these nsidered independently when determining the MAGNITUDE and the stresses or forces. This is important to remember when the NS are combined as in diagram 2. In determining the CHARACTER *AC*, for example, from the STRESS-POLYGON *dac* for joint 1, the owards joint 1, while from the STRESS-POLYGON *abc* for joint 2, *ca* oint 2. In both cases the piece *AC* is pushing against the joints is in COMPRESSION. If arrow-heads are used in indicating the di- forces in the STRESS-POLYGONS, they should be erased as soon as of the stresses for the joint being considered have been found; re polygons are combined as in diagram 2, *each line will have two* inting in *opposite directions, leading to confusion.* Arrow-heads *upon the* TRUSS-DIAGRAM. *Each piece will have two arrow-heads, referring to the joint at the end. When the arrow-heads point*

away from each other the piece is in COMPRESSION, and wh
towards each other the piece is in tension.

It is important to keep in mind the direction in which the forc
are considered in order, in going around the truss or around a j
6 and 8 the curved arrows show that a clockwise direction ha
This makes the stress-lines of the stress-diagram come on the le
line This direction has been taken for all the trusses in this
for a few diagrams for wind-loads
could have been determined ju
taking a contra-clockwise direction

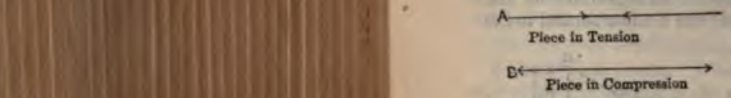
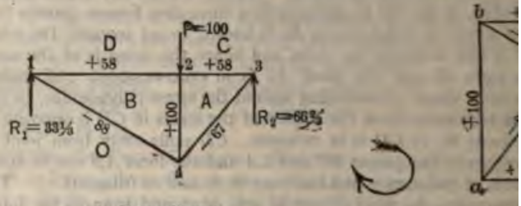


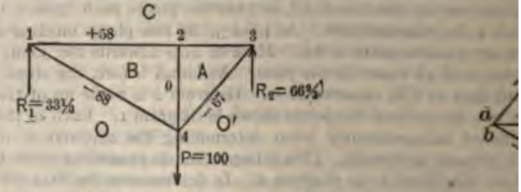
Fig. 7. Indication of Character of Stress

If two men pull on the two
exerting PULLING FORCES of equa
TENSIONAL STRESS in every cross
rope is equal to the FORCE with
pulls; and each end of the rope p
the man holding it, with a FORCE equal in magnitude to that w
Thus if each man exerts a FORCE of 100 lb the STRESS in the rop
each end of the rope pulls away with a FORCE of 100 lb. If the m
the two ends of a piece of timber with a FORCE of 100 lb, the
against each man with a FORCE of 100 lb, although the enti
STRESS in every cross-section of the timber is but 100 lb.
STRESS-LINES are sometimes drawn with arrow-heads pointin
other, as at *A*, Fig. 7, denoting TENSION; or with arrow-



1. Truss-diagram

2. St

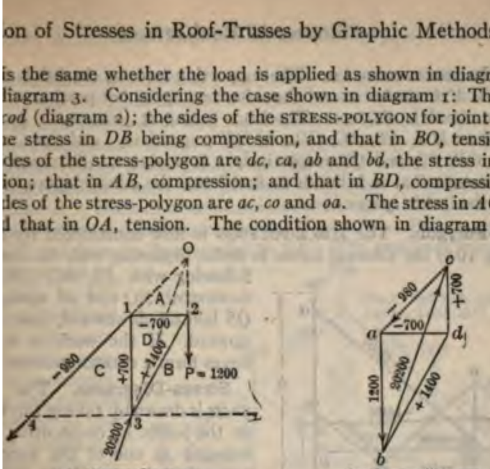


3. Truss-diagram

4. St

Fig. 8. Trussed Beam

is the same whether the load is applied as shown in diagram 1, or
liagram 3. Considering the case shown in diagram 1: The FORCE-
:od (diagram 2); the sides of the STRESS-POLYGON for joint 1 are *od*,
ie stress in *DB* being compression, and that in *BO*, tension. For
des of the stress-polygon are *dc*, *ca*, *ab* and *bd*, the stress in *CA* be-
ion; that in *AB*, compression; and that in *BD*, compression. For
des of the stress-polygon are *ac*, *co* and *oa*. The stress in *AC* is com-
l that in *OA*, tension. The condition shown in diagram 3, where



1. Truss-diagram 2. Stress-diagram

Fig. 9. Crane Truss

ıspended from joint 4, leads to a different form of STRESS-DIAGRAM,
hod of construction remains the same. The stresses in the pieces
with the exception that the stress in the piece *AB* is zero for the case
ıgram 3.

ie Truss. Fig. 9, diagram 1, shows the TRUSS-DIAGRAM of a CRANE
ertical load at joint 2. The EXTERNAL FORCES acting on the frame
l at joint 2, the supporting force at joint 3, and the stress in the
nce the frame is in equilibrium under the action of these three forces,
in a point.
les a ready
determining
ı and magni-
e supporting
: 3. Prolong
and draw a
ne through
it intersects
prolonged, at
ıce the sup-
e must pass
nt 3, 3*O* is
on of this
FORCE-POLY-
lrawn. The sides of this polygon are *bc*, *ca* and *ab*. *ca* is the stress
'*A* which is in tension. The stress-polygons for each joint can now
lrawn and the stresses in the members of the frame determined
.

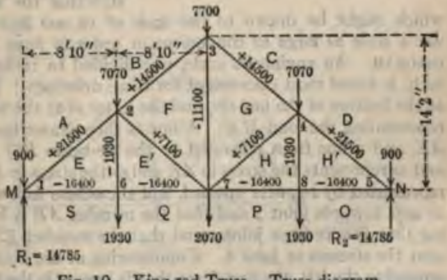


Fig. 10. King-rod Truss. Truss-diagram

ving EXAMPLES, worked out in detail and with considerable repeti-
able the student to grasp the principles of the GRAPHIC METHOD
ing STRESSES IN *FRAMED STRUCTURES*.

Truss. Example 1. Fig. 10 shows the TRUSS-DIAGRAM of the
ted in Fig. 1, properly drawn, lettered and figured, ready for

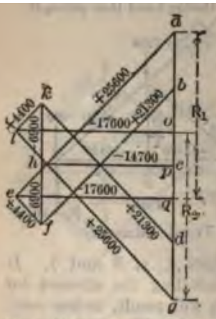
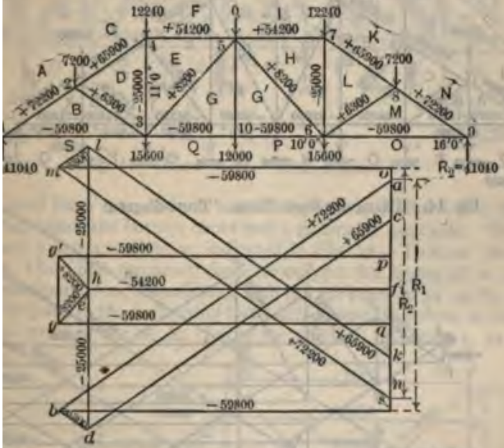


Fig. 12A. Queen Truss.
Stress-diagram

parallel to EQ. The triangle
three forces in equilibrium, mee
acting at joint 1. As the su
upward, the arrow-head on *qa* p
lowing the sides of the stress-pol
ae acts towards the joint and
showing that *ae* is in compressio
Next determine the stresses acti
stress in *EA* is now known and
line *ea*, and as the stress at joint
opposite to that at joint 1, it
towards joint 2. The next forc
lb, which acts downward. The
been found by measuring from *a*
6 300 lb at the same scale as
There now remain two stresses t
2, those in *BF* and *FE*. Draw *bf*
fe parallel to *FE*, the two line
Then the sides of the polygon *abfe* represent respective
of the four forces acting at joint 2; and the characte
determined by the directions in which the stress-lines ar
in going around the joint. In this case they all act
and *EA*, *BF* and *FE* are in compression. The stresses ac
may be determined next, as only two of them are unkn
Considering the external force and the three stresses acting
in *FB* has been determined and is represented by the line *fb*
ward for this joint. The load or force *BC*, 8 550 lb, is know
by *bc*. *ch* is drawn parallel to *CH*, and *hf* parallel to *HF*,
The length of *ch* determines the magnitude of the stress in C
in *HF*. The stresses in all the truss-members but *HP* a

le of not less than ⅛ in to the foot; and the stress-diagram should
ine by line, in accordance with the foregoing directions and the
ined and compared with those given in the figures. A variation of
lb for small stresses and less than 1% for large stresses may be ex-
a greater variation indicates either that sufficient care has not been
drawing the stress-lines exactly parallel to the corresponding lines
-diagram, or that an error has been made in drawing the truss-dia-
scaling the lines of the stress-diagram. After these two examples
orked, a number of the following examples, also, should be solved,
nciples are fully understood.

r Museum of Fine Arts, St. Louis, Mo. **Example 3.** Fig. 13
he truss-diagram of the truss shown in Fig. 11, Chapter XXVI,



ig. 13. Truss-diagram. Museum of Fine Arts, St. Louis, Mo.
Fig. 13A. Stress-diagram

dicated being approximately those due to the roof and suspended
The loads being symmetrically disposed, each supporting force is
-half the total load, or 41 040 lb. The counterbraces *CC*, shown
XXVI, are omitted from the truss because they have no stress when
uniformly loaded. To draw the stress-diagram (Fig. 13A), first
e the vertical line *sa*, equal to 41 040 lb, equal to R_1; and then *ab*
llel respectively to *AB* and *BS* and representing the stresses acting
At joint 2, the line *ba* represents the stress in *BA*; *ac*, equal to 7 200
AC; *cd*, the stress in *CD*; and *db* the stress in *DB*. The polygon
ents the forces in equilibrium acting at joint 2. At joint 3 there are
wn forces; and as three unknown forces out of five in one polygon
etermined, joint 4, where *dc* and the load *CF* are known, is considered
suring off the load *cf*, equal to 12 240 lb, the stresses in *FE* and *ED*
be determined. These are found by drawing *fe* parallel to *FE*, and
ED, the two lines intersecting at *e*. At joint 3, *sb*, *bd*, *de* and the
own, and *eg* and *gq* are drawn to close the polygon *sbdegs*. At

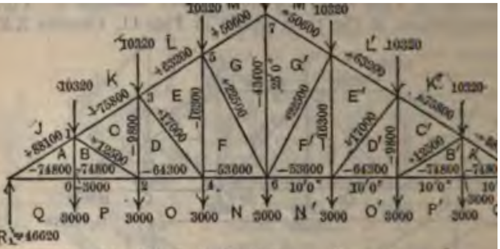
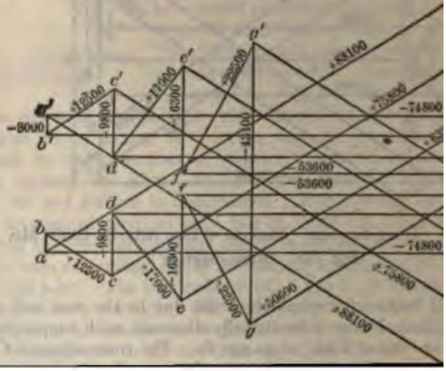


Fig. 14. Triangular Pratt Truss. Truss-diagram



the polygon. At joint 5, *fe*, *el* and *lm* are known and *mg* and *gf* int 7 is considered next, for at joint 6 there are three unknown y the graphic method three out of five forces, meeting in a point ium, must be known in order to determine the other two. At *mm'* are known and *m'g'* and *g'g* are drawn to close the polygon. the determination of the stresses in all the pieces for one-half d of course the stresses for each half are the same as the loading .

l Howe Truss. Example 5. For the next example a Howe dered, whose center lines give the diagram shown in Fig. 15.

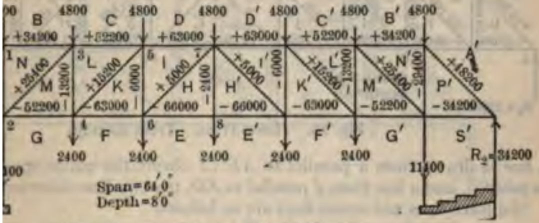


Fig. 15. Howe Truss. Truss-diagram

sed for a span of 64 ft, and it supports, in addition to a flat roof, g below the bottom chord and a gallery on each side. The loads t joints are about as indicated in Fig. 15. To draw the stress- 15A), first construct the force-polygon by laying off to scale in ternal forces, commencing with the left reaction 34 200 lb. Next, t joint 0, the supporting force *sa* is known, the stress in the rafter stress in the tie *ps*, closing the polygon. At joint 1, *pa* and *ab*

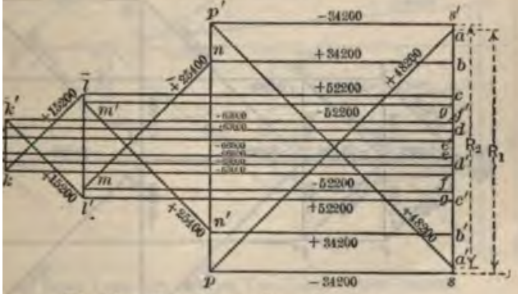


Fig. 15A. Howe Truss. Stress-diagram

bn and *np* are drawn, closing the polygon. At joint 2, *gs*, *sp* and and *nm* and *mg* are drawn. At joint 3, *mn*, *nb* and *bc* are known are drawn. The stresses at the remaining joints are found in the hose at 3 and 4. The stresses in pounds in the various members noted in figures in the *stress-diagram (Fig. 15A).*

s Loaded at Alternate Joints. Example 6. *(Fig. 16.)* This *OWE TRUSS is selected to show how to proceed* when there is no load at one or more of the joints. Fig. 16 represents the center lines 50 ft in span and only 5 ft in height. In order to give the braces an approximating 45°, the truss is divided into ten panels; but purlin over every other joint, as in Fig. 16, Chapter XXVI. The loads purlins are about 5 000 lb. The stresses at joint 1 are found in the sa as in the previous example, always starting with the supporting joint 2 the stress-line *da* is already drawn; and as there is no load a

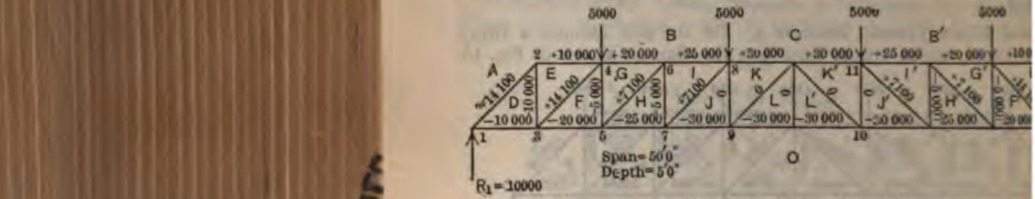


Fig. 16. Howe Truss. Truss-diagram

a line is drawn from *a* parallel to *AE* (*A* covers the entire space f to joint 4), and a line from *d* parallel to *ED*, the two lines intersectin

The force-lines and stress-lines are as follows:

At joint 3: *od*, *de*, *ef* and *fo*;
At joint 4: *fe*, *ea*, *ab*, *bg* and *gf*;
At joint 5: *of*, *fg*, *gh* and *ho*;
At joint 6: *hg*, *gb*, *bi* and *ih*;
At joint 7: *oh*, *hi*, *ij* and *jo*;
At joint 8: *ji*, *ib*, *bc* and *ck*;

the latter line extending to the point of beginning, *j*, showing that stress in *kj*. At joint 9 the only stresses are *oj* and *lo*, for as there

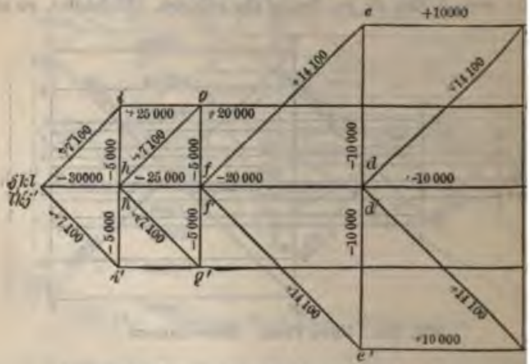
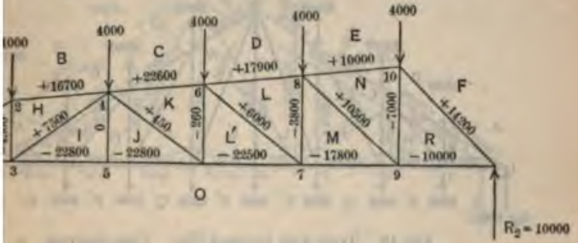


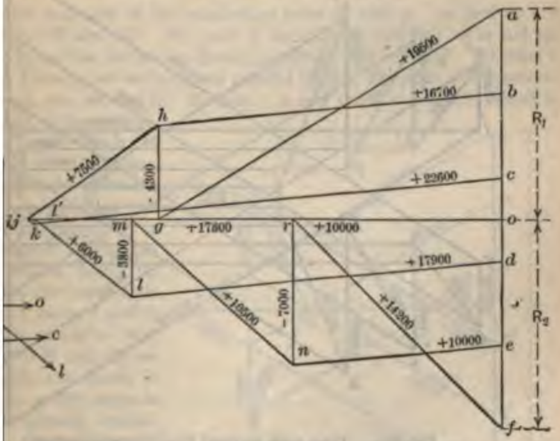
Fig. 16a. Howe Truss. Stress-diagram

in *JK*, for equilibrium there can be none in *KL*. There is, also, no s middle rod. Although these members have no stress, it is advisabl them in the truss in order to stiffen the top and bottom chords. be made very light, say ½ in in diameter for the rods and 3 by 6 section for the braces.

uss with Slanting Top Chord. **Example 7.** In order to give a
roof it is often desirable to incline the top chord of a Howe truss
Chapter XXVI. Fig. 17 shows the truss-diagram for such a truss,
the stress-diagram. The latter is drawn in the same way as the
m in Example 5, but because the top chord is not level, the stress-
ot symmetrical. When the stress-diagram is not symmetrical it is
complete the entire diagram, so as to show the stress in every mem-
ss. The stress-lines for joint 9 are *om*, *mn*, *nr* and *ro*. This leaves



g. 17. Howe Truss with Slanting Top Chord. Truss-diagram



17A. Howe Truss with Slanting Top Chord. Stress-diagram

rf to complete the diagram; and if the diagram has been correctly
e joining *r* and *f* will be exactly parallel to *RF*. There is no stress
.

th Inclined Ties. **Example 8.** (Fig. 18.) This truss has the
ions as the truss shown in Fig. 14, but the diagonals *incline in the*
ction and *are in tension, while the verticals, except* the middle one,
mpression. This form of truss is sometimes used in wooden con-
oid the long middle braces shown in Fig. 14. Long ties are, as a

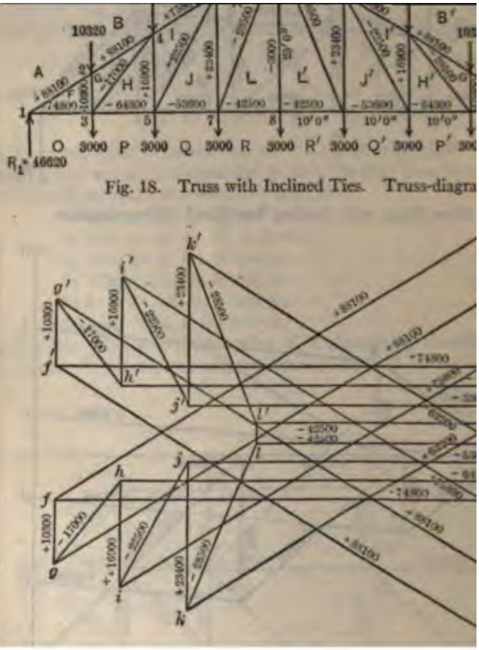


Fig. 18. Truss with Inclined Ties. Truss-diagra

Fink Truss. **Example 10.** (Fig. 20.) The inclination of the
and the distance between the trusses 20 ft. The loads are cal-
slate roof on boards or on angle-iron purlins. Commence the
by drawing a vertical line equal to the supporting force R_1, or
lettering the lower end of the line *o* and the upper end *a*, as these

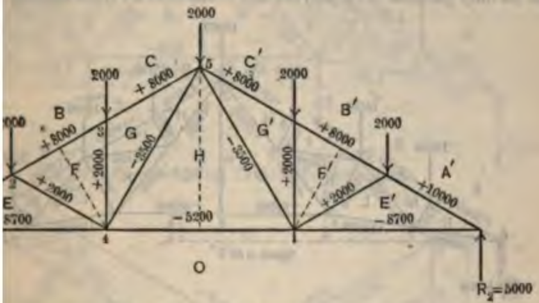


Fig. 19. Fan Truss. Truss-diagram

on each side of the supporting force at joint o. *an* and *no* are
to *AN* and *NO*. For joint 1, *na* is drawn upward; *ab* is laid off
o lb and *bm* and *mn* are drawn parallel to *BM* and *MN*. At joint 2
e known, and *ml* is drawn parallel to *ML*, the sides of the stress-
on, *nm*, *ml* and *lo*. At joint 3 a new condition is met, which is
ny of the preced-
and which is
his form of truss,
parently unknown
a study of the
however, it is
I and *IK* act as
Y-RODS, taking up
m the lower ends
at joints 2 and 5;
ds at joints 1 and
d *NM* and *IH* are
th, the stress in
ne as the stress in
s already known.
the number of
es at joint 3 to
t force known at
, the next *mb* and
qual to 16 100 lb.
e is drawn parallel to *CI* and from *I*, the initial point, a line
. Between these two lines there must be a line, *ik*, parallel to
in length to *ml*; and this line is determined by means of the
parallel ruler and *straight-edge*. *If correctly drawn, the joint i*
with mm. The sides of the stress-polygon for joint 3 are, then,
t and kl.

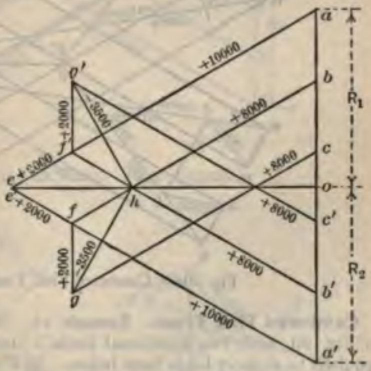


Fig. 19A. Fan Truss. Stress-diagram

At joint 4 the stress-lines are *ol*, *lk*, *kg* and *go*.
At joint 5 the stress-lines are *gk*, *ki*, *ih* and *hg*.
At joint 6 the stress-lines are *hi*, *ic*, *cd* and *dh*.

If the stress-diagram is accurately drawn, a line from *d* parallel to the will pass through the point *h*. The vertical tie *GG′* (Fig. 20) has no and its only purpose is to prevent the horizontal tie from sagging.

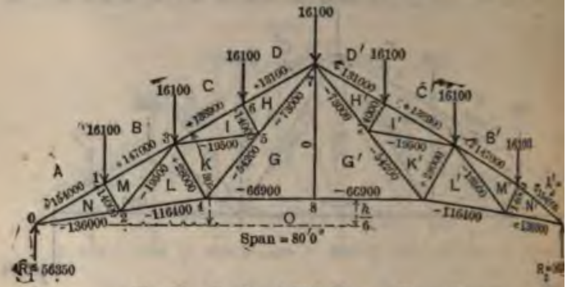


Fig. 20. Cambered Fink Truss. Truss-diagram

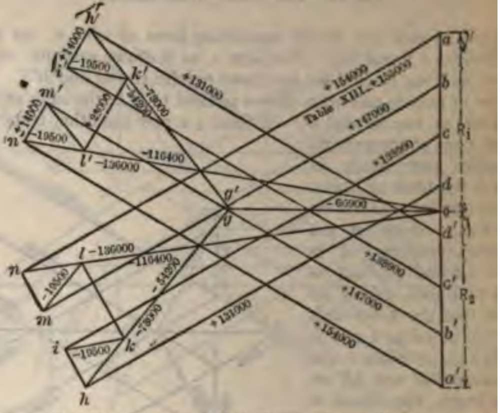


Fig. 20A. Cambered Fink Truss. Stress-diagram

Cambered Fink Truss. **Example 11.** (Fig. 21.) This is the tru in Fig. 20, with two additional loads. Steel trusses of this shape a required to support loads from below. In Fig. 21 there are two loads each, supported at joints 5 and 9, in addition to the roof-loads. Th diagram is drawn in exactly the same way as in Fig. 20A, except that the first-known force *ro*, parallel to *RO*, 4 tons, is laid off, locating *r*. joint, then, *ro*, *ol* and *lk* are known and *kg* and *gr* are drawn to close the It should be noticed that the stresses in *NM*, *IH*, *ML*, *KI* and *LK* are as the stresses in the corresponding members of Fig. 20, as these are not by the ceiling-load. All of the other stresses, however, are increased the increase in the supporting forces, the greatest increase being in

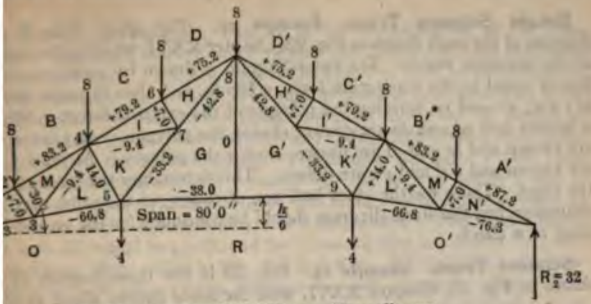


Fig. 21. Cambered Fink Truss. Truss-diagram

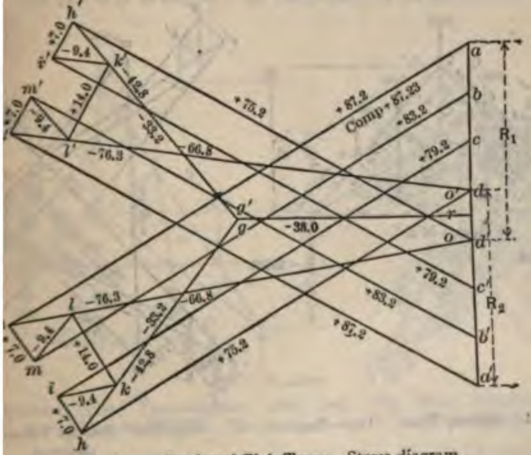
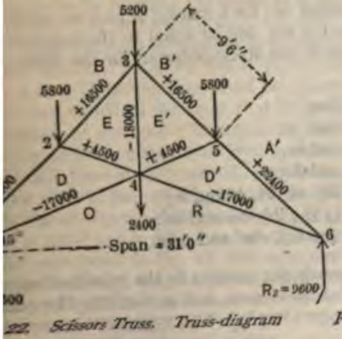


Fig. 21A. Cambered Fink Truss. Stress-diagram



22. *Scissors Truss. Truss-diagram*

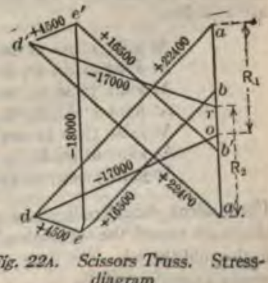


Fig. 22A. Scissors Truss. Stress-diagram

Simple Scissors Truss. Example 12. (Fig. 22.) Th
diagram of the truss shown in Fig. 22, Chapter XXVI, which is
of the SCISSORS TRUSS. The truss-diagram is drawn by com
line *oa* equal to the supporting force, 9 600 lb, and then in or
bb', *b'a'*, *a'r* and *ro*, forming the polygon of the external force
is known and *ad* and *do* are drawn, closing the polygon. At j
are known and *be* and *ed* are drawn, closing the polygon. At
are known and *b'e'* and *e'e* are drawn. This determines the st
the truss. Those for the other half are, of course, of the sam
character, but the stress-diagram should be continued for the
truss as a check.

Scissors Truss. Example 13. Fig. 23 is the truss-diag
shown in Fig. 23, Chapter XXVI, with the loads figured abo

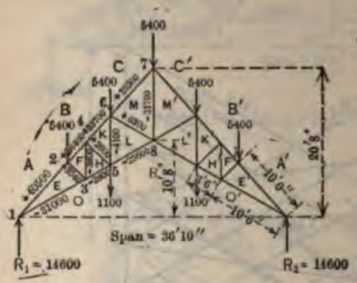


Fig. 23. Scissors Truss. Truss-diagram

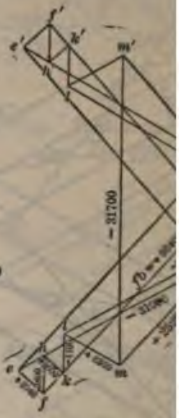


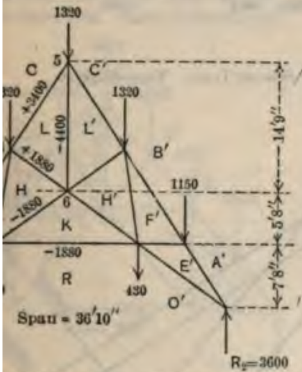
Fig. 23A. Sci

be for a slate roof and wooden ceiling and for a spacing of 12 f
stress-diagram is begun by drawing the line *oa* equal to th
at joint 1 (14 600 lb). The sides of the stress-polygons for
are as follows:

At joint 1: *oa*, *ae* and *eo*;
At joint 2: *ea*, *ab*, *bf* and *fe*;
At joint 3: *oe*, *ef*, *fh* and *ho*;
At joint 4: *hf*, *fb*, *bk* and *kh*;
At joint 5: *ro* (1 100 lb), *oh*, *hk*, *kl* and *lr*;
At joint 6: *lk*, *kb*, *bc* (5 400 lb), *cm* and *ml*;
At joint 7: *mc*, *cc'* (5 400 lb), *c'm'* and *m'm*.

The student should notice how much the stresses in the pri
this truss exceed the supporting forces or loads, and particula
in the middle rod. For these reasons this is not an econom
spans exceeding 36 ft.

ʼruss. **Example 14.** Fig. 24 is the truss-diagram of the truss
4, page 1056, and for which the roof and ceiling-loads are computed
, page 1055. The truss shown in Fig. 4 is built of planks spiked
gether, but the stresses are found in precisely the same way if the
of heavy timbers and supports a greater roof-area. It should be
hat only the shape of the truss and the loads, including their point
, affect the stress-diagram. The stresses at joints 1 and 2 are
, commencing with *oa*, equal to R_1. At each of joints 3 and 4,
e are three unknown forces. We cannot obtain the stresses at
hose acting at joint 3 have been determined. The known forces
ad *RO*, equal to 430 lb, and the stresses acting in *OE* and *EF*; and
forces, those acting in *FH*, *HK* and *KR*. *HK* and *KR* are in ten-
e to hold joint 3 from falling down and outwards. Either one,
may be omitted, and the greater the stress is in one the less it is in
he only way to complete the stress-polygon for joint 3 is to fix the



ors Truss. Truss-diagram. (See, also,
and Chapter XXVIII, Fig. 2)

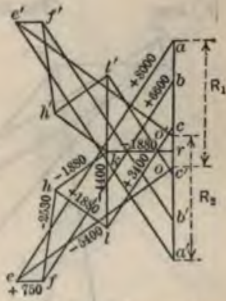


Fig. 24A. Scissors Truss.
Stress-diagram

ne of the unknown stresses arbitrarily. The most satisfactory
s to be to make the stress in *HK* equal to that in *KR*. This is
s: The first known force at joint 3 is the load represented by *ro*,
ing obtained by measuring upwards from *o*, 430 lb; next, the lines
known. From *f* a line is drawn parallel to *FH* and from *r*, a line
R. These two lines must be connected by a third line parallel to
ne should be drawn so that its length is equal to *kr*, which can be
ns of dividers. Lettering the ends of this line *h* and *k* the sides
ted stress-polygon for joint 3 are *ro*, *oe*, *ef*, *fh*, *hk* and *kr*. Knowing
F, there are but two unknown forces at joint 4, and these are readily
sides of the stress-polygon for joint 5 are *lc*, *cc′*, *c′l′* and *l′l*. Com-
ress-diagram with that of Fig. 23A, it is seen that the stress in the
much less in proportion to the loads for the truss shown in Fig. 24
ne shown in Fig. 23, this reduction being due to the horizontal tie
ht trusses built of planks, spiked or bolted together, the form of
Fig. 24 is *preferable to that shown in Fig. 23.*

russ. Example 15. *Fig. 25 is the truss-diagram of the* SCISSORS
Fig. 27 of Chapter XXVI. The line EF in Fig. 25 does not

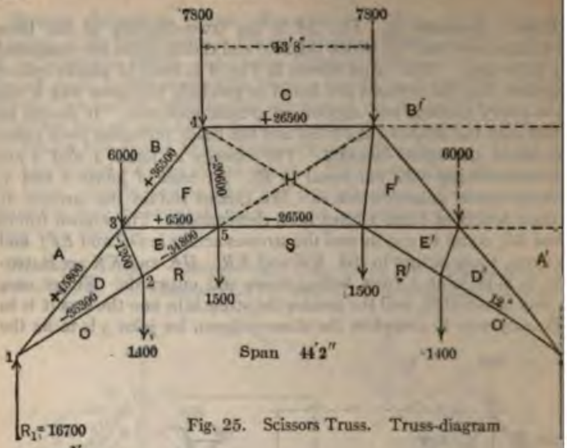


Fig. 25. Scissors Truss. Truss-diagram

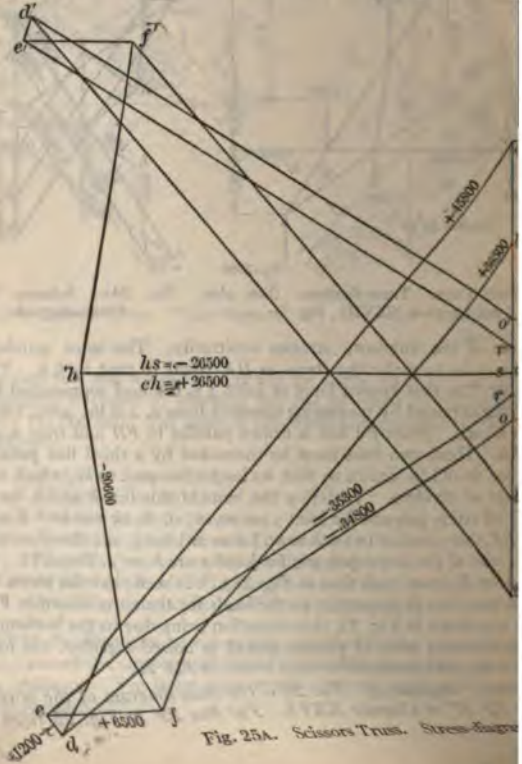
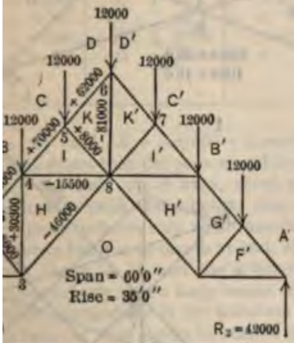


Fig. 25A. Scissors Truss. Stress-diagra

with the center line of the strut in Fig. 27, because the inner end of dropped slightly on account of the detail of the joint; but in truss- lines must go from joint to joint, otherwise the stress-diagram can ı. There are no stresses in the middle diagonals under a symmetrical ; hence they are shown by dotted lines in Fig. 25. As no complica- n drawing the stress-diagram of this truss, a detailed description is . The sides of the stress-polygons for the different joints are as

For joint 1: *oa, ad, do*;
For joint 2: *ro, od, de, er*;
For joint 3: *ed, da, ab, bf, fe*;
For joint 4: *fb, bc, ch, hf*;
For joint 5: *sr, re, ef, fh, hs*.

incide, showing that the compression in *CH* is equal to the tension e plus and minus signs in Fig. 25, as in all the other diagrams, ression and tension respectively.



Truss without Tie-beam. Truss-diagram

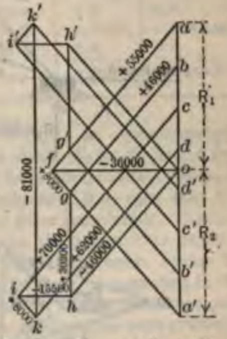


Fig. 26A. Truss without Tie-beam. Stress-diagram

ithout Tie-Beam. **Example 16.** Fig. 26 shows a truss which is ISSORS TRUSS nor a HAMMER-BEAM TRUSS, yet this form can be made milar to the HAMMER-BEAM TRUSS by inserting a curved brace below replacing the pieces *OH* and *OH′* by curved members. There is no drawing the stress-diagram shown in Fig. 26A.

izontal Thrust of Scissors Trusses. In the examples just given assumed that the reactions are vertical and consequently that there ONTAL THRUST. This would be true if the materials composing the absolutely rigid. This is not the case, however, and all trusses the geometrical lines of their shape change in shape after the full lied. In the SCISSORS TRUSS this changes the length of the span, onger and permitting the rafters to sag. If the trusses are con- h a camber in the rafters and the span made a little short, the THRUST supports can be practically eliminated by fastening one end of the oviding for a movement at the other, so that when the full roof and have been placed on the truss the span will have its correct length. this we must know HOW MUCH THE SPAN WILL CHANGE IN LENGTH oad. This can be determined in the manner shown in the follow-

ing example and by referring to Fig. 26B. Let Diagram 1 represent a
SCISSORS TRUSS loaded as shown with 1 000 pounds at each top-chord
and let the left end be assumed to rest upon rollers. Then both reactions
be vertical and the stresses in each member can be found from the usual
diagram shown in Diagram 2. Let S be the stress in any member as
from Diagram 2; u, the stress in any member produced by one pound
horizontally at K and towards L as found from Diagram 3; A, the area of

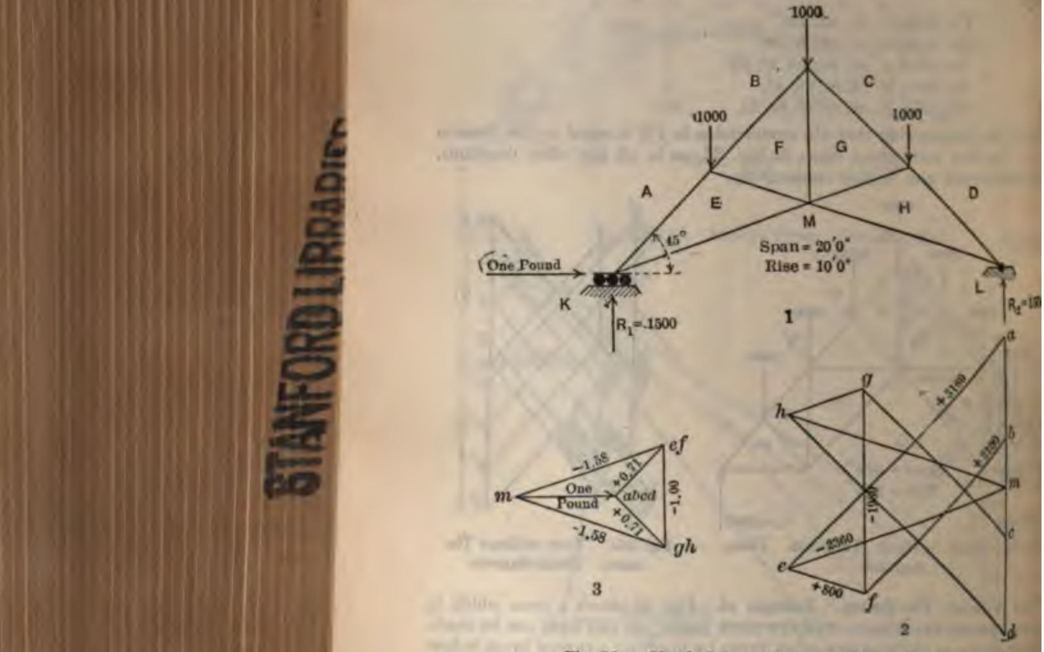


Fig. 26B. Simple Scissors Truss and Stress-diagrams

member, in square inches; l, the length of any member, in inches; E,
modulus of elasticity of the material composing any member and D, the
CHANGE IN LENGTH OF SPAN when the truss is subjected to its full load.

$$D = \sum \frac{Sul}{AE}*$$

If H is the HORIZONTAL FORCE applied at K, which is necessary to make
value of $D = 0$

$$H = D \div \sum \frac{u^2 l}{AE}$$

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* Theory and Practice of Modern Framed Structures, Johnson, Bryan and
(John Wiley & Sons); Roofs and Bridges, Merriman and Jacoby (John

calculations for Fig. 26B are given in Table XVII, assuming that , excepting *FG*, are composed of 6 by 6-in white pine timbers with 00 lb per sq in, and that *FG* is an upset round steel rod having .785 sq in with *E* equal to 30 000 000 * lb per sq in for steel.

II. Computations for *D* and *H* for a Particular Scissors Truss

(2) S, Diagram 2	(3) A	(4) $S \div A$	(5) u, Diagram 3	(6) l	(7) $\frac{Sul}{AE}$	(8) $\frac{u^2l}{AE}$
+3160	36	87.8	+0.71	84.8	0.00528	0.00000118
+2100	36	58.3	+0.71	84.8	0.00351	0.00000118
+2100	36	58.3	+0.71	84.8	0.00351	0.00000118
+3160	36	87.8	+0.71	84.8	0.00528	0.00000118
−2360	36	65.5	−1.58	126.5	0.01316	0.00000875
−2360	36	65.5	−1.58	126.5	0.01316	0.00000875
+ 800	36	22.2	0	63.2	0.0	0.0
−1980	0.785	25.22	−1.00	80.0	0.00672	0.00000340
+ 800	36	22.2	0	63.2	0.0	0.0
					0.05062	0.00002562

in and $H = 0.05062 \div 0.00002562 = 1975$, or, approximately, his shows that the span would lengthen about $\frac{1}{20}$ in, if allowed free t one end; or, if fixed, there would be a HORIZONTAL FORCE of ding to push the supports out. In column 4 it is seen that the square inch are only about one-tenth of those permissible. Assum- loads become 10 000 lb at each apex-joint, the HORIZONTAL DEFLEC- es about ½ in, and the HORIZONTAL THRUST becomes 20 000 lb. onclusively that a large excess of material must be employed in the ss, particularly in the members EM and HM which contribute over value of *D* as shown in column 7, if the HORIZONTAL DEFLECTION is so small that its effect may be neglected. As stated before, if the itted to deflect horizontally until fully loaded, the walls or supports sibly no HORIZONTAL THRUST to resist.

mer-Beam Truss. As usually constructed the HAMMER-BEAM ected to exert more or less HORIZONTAL PRESSURE at the supports; rovided for by heavy walls and buttresses. The diagram of such own in Fig. 27, in which the CURVED BRACES usually built in the of the truss are not shown, as they are considered to be purely orna- for vertical loading have no stresses. The brace *OM* is drawn as re straight; but a curved brace may be used instead, without alter- ram. The stress in the curved piece is that found from the stress- reased by the bending stress due to its curvature. To determine e members of this truss it is necessary to first find the HORIZONTAL e truss against the wall. To do this all the truss-members from int 4 are considered to form a FRAMED BRACE, or ASSEMBLAGE OF rting the upper portion of the truss at joint 4, or a SINGLE BRACE, e broken line 04, Fig. 27, is assumed to have the same effect on the e pieces put together *in the FRAMED STRUT; that is,* the truss is

o lb per sq in is used for the value of E for steel the values of *D* and *changed. See Table I, page 664.*

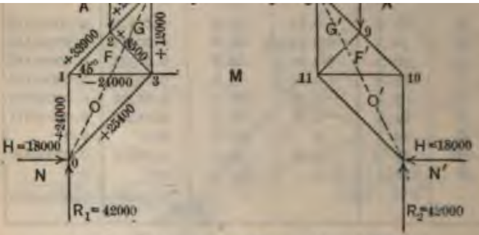
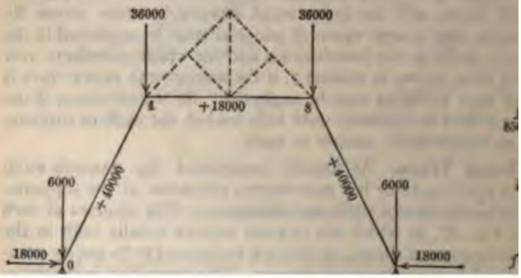


Fig. 27. Hammer-beam Truss. Truss-diagram

two lines will intersect at *m*, and *mx* is the MAGNITUDE OF THRUST exerted on the wall at the joint o. Having obtain is easy to determine the stresses in the pieces. At joint o



at
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gon. It will be noticed that the POLYGON CLOSES without allow- be drawn parallel to *IM*; hence there is no stress in *IM*, with . When there are wind-loads there is some compression in *IM*, r is a necessary part of the truss. At joint 5 there are the stresses and at joint 6. *kd*, *dd′*, *d′k′* and *k′k*, which complete the stresses for ss, which are all that are needed. Comparing, now, the diagram

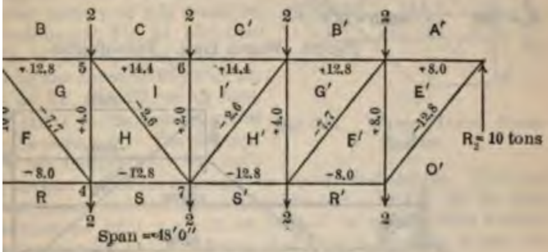


Fig. 28. Suspended Pratt Truss. Truss-diagram

27B, with Fig. 26A, it is seen that, in general, the MAGNITUDES s in the truss, Fig. 27, are much less than those of the stresses g. 26; while, on the other hand, the latter truss may be so con- to exert the OUTWARD THRUST on the walls, exerted by the truss 27. If curved members are introduced between joints 3 and 2, they should be lightly secured at the ends or the stresses deter- manner SCISSORS

d Pratt ple 17. be the of a sus- T TRUSS, ed at top ith 2 tons t. The is drawn same way E TRUSS, diagonals site direc-

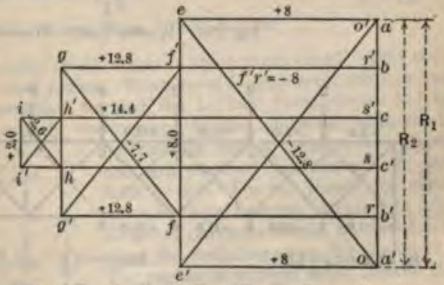


Fig. 28A. Suspended Pratt Truss. Stress-diagram

sses should be drawn for the joints in the order in which they In this truss the verticals are in compression and the diagonals

ıss. Example 18. Fig. 29 is the truss-diagram of a light iron of 32-ft span, intended to support a tar-and-gravel roof, the right-angles to the line of the trusses. The stress-diagram is previous examples, taking the joints *in the order in which they*

ren Truss, or Lattice Truss. Example 19. The truss-dia- *ig. 30 is best analyzed by considering* it as built up of two

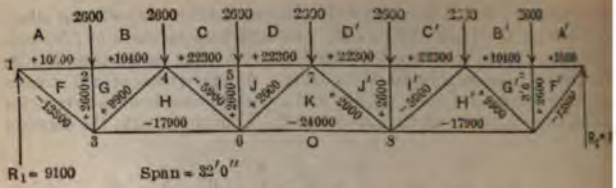


Fig. 29. Warren Truss. Truss-diagram

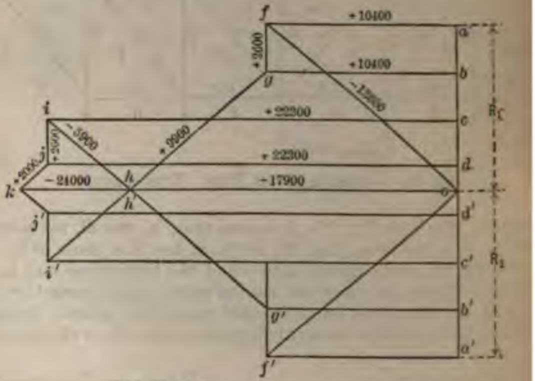


Fig. 29A. Warren Truss. Stress-diagram

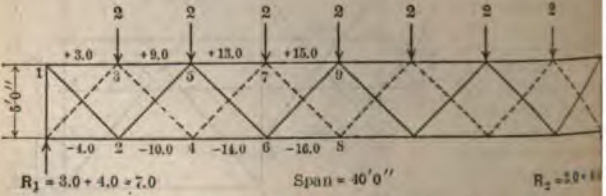


Fig. 30. Double Warren Truss. Truss-diagram

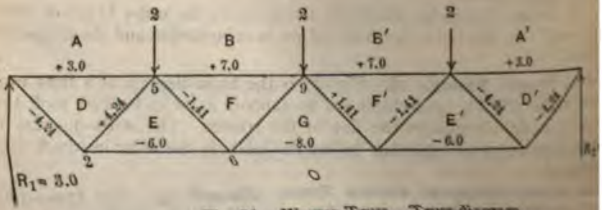


Fig. 31. Warren Truss. Truss-diagram

sses, laid one over the other, the full lines indicating a truss such Fig. 31, and the dotted lines a truss as shown in Fig. 32. Three oads would come on the first truss and four on the second. The ound for each truss separately ibined for the top and bottom s the stress in the top chord Fig. 30, would be that in *AD*, tons; from 3 to 5 it would be stress in *AD*, Fig. 31, plus that 32, or 9 tons; from 5 to 7 it al to the stress in *BF*, Fig. 31, s in *BE*, Fig. 32, or 13 tons, he stress in the bottom chord 1 the same way. The diagonal es act independently of each e stresses are those indicated on the stress-diagrams. The plus compression and the minus signs tension. In Fig. 32 the sides polygon for joint 7 are *fe*, *eb*, *bc*, and *ge*, which closes without a line parallel to *GF*, showing that there is no stress in the two s except that due to the weight of the bottom chord. This truss structed of steel angles. When wood is employed, three or four sses are combined forming the LATTICE TRUSS shown in Fig. 8. It is entirely unnecessary to use graphical methods in determining the stresses, as the chords and web-members are respectively uniform in size. For the chords the maximum bending moment divided by the distance, center to center, of the chords gives the designing-stress for the chords. The maximum vertical shear, usually the reaction, divided by the number of simple WARREN TRUSSES combined gives the vertical component for which the web-planks are designed. This, of course, leads to a waste of material as far as resisting stresses is concerned, but for stiffness and economy in labor, the extra ell used. This truss can be extended indefinitely by giving it it for the span. (See, also, pages 1008 and 1009.)

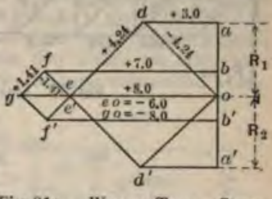


Fig. 31A. Warren Truss. Stress-diagram

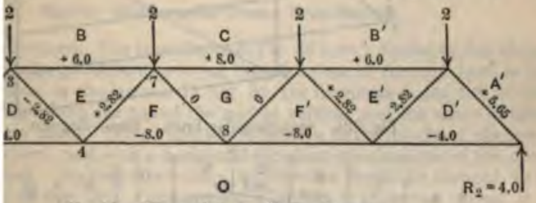
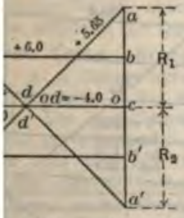


Fig. 32. Warren Truss. Truss-diagram



rren Truss. Stress-diagram

lar Truss. *Example 20. Fig. 33 is the truss-diagram of a truss hown in Fig. 59, Chapter XXVI, the panel-loads being taken at the analysis being the same for any other loads.* The stress-

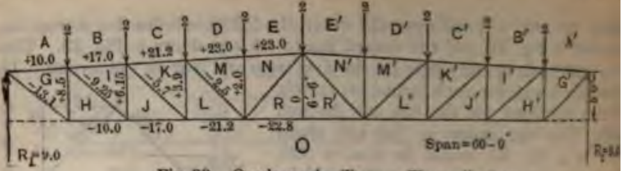


Fig. 33. Quadrangular Truss. Truss-diagram

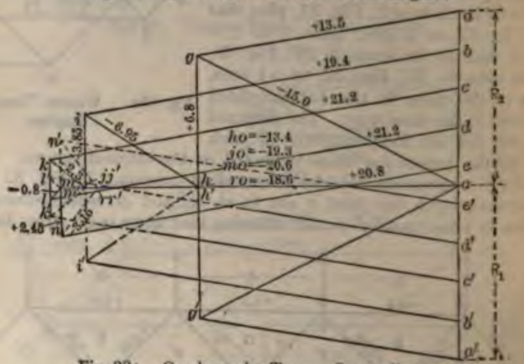


Fig. 33A. Quadrangular Truss. Stress-diagram

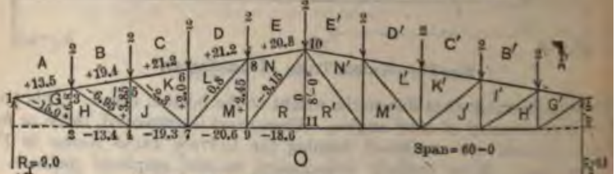


Fig. 34. Quadrangular Truss. Truss-diagram

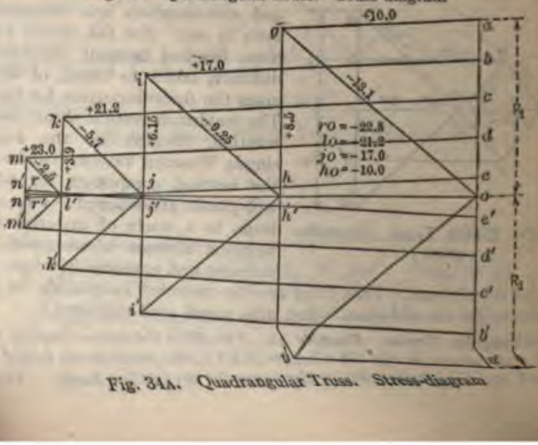


Fig. 34A. Quadrangular Truss. Stress-diagram

s drawn exactly as in the previous examples, commencing with the g force *oa* and considering the joints in the order in which they are . In this truss the diagonal web-members are all in tension and the n compression. It will be noticed that the inclination of the diagonals panels nearest the middle of the truss is opposite to that of the diagonals er panels. This is due to the inclination of the top chord, which causes ve stresses in the inner diagonals when they incline the other way. in *LM*, however, is so small that a single steel angle resists either a com-

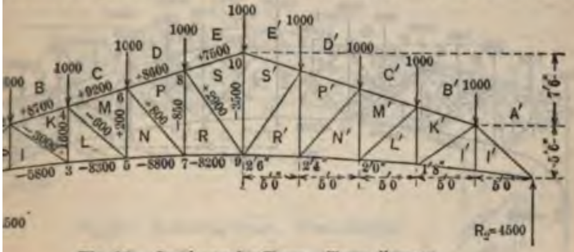


Fig. 35. Quadrangular Truss. Truss-diagram

r tensile stress. The truss shown in Fig. 34 is very similar to that shown , the principal difference being that the slope of the top chord is less mer than in the latter. In Fig. 34, the diagonals in the two middle ly incline from the top of the middle vertical, and the stress in these is very small. With a still less inclination to the top chord, the stress omes zero; and with a horizontal top chord the character of the stress reversed. To keep it in tension, its direction should be changed, as in T TRUSS, Fig. 28. Comparing the stresses in these two trusses, it is while the stresses in the s are less in Fig. 34A than in the stresses in the chords at le are considerably greater. e, the less the height of a proportion to the span, the he stresses in the chords, at the middle of the truss.

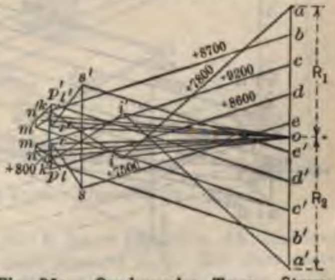


Fig. 35A. Quadrangular Truss. Stress-diagram

angular Truss. Example 35 is a truss-diagram the truss shown in Fig. 66, XXVI. The truss-diagram s in the previous examples, at in this case, as the hord has different inclina- he different panels, the stress-lines do not lie over each other, but m *o*, the lines in the stress-diagram being parallel to the corresponding he truss-diagram. In this truss the character of the stresses in the web-members is reversed in the two panels nearest the middle. Thus f the stress-polygon for joint 4 are *lk*, *kb*, *bc*, *cm* and *ml*, the stress *ml* m the joint and hence denoting tension. At joint 8 the sides of the gon are *rp*, *pd*, *de*, *es* and *sr*, *the latter line acting towards the joint, denoting compression. Under irregular loading,* the character of *SR would probably be reversed,* so that the piece would be in ten-

sion instead of in compression. The st
34 and 35 should, therefore, always b
truss only and also for wind-pressure.

Quadrangular Truss. **Example 2**
the truss illustrated in Fig. 65, Chap
shown in Fig. 34, except for the seco

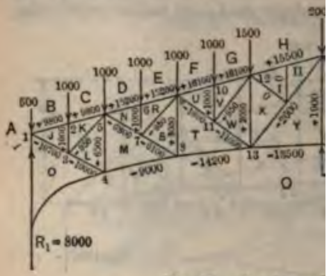


Fig. 36. Quadrangul

curved bottom chord. The stress-dia
the lines from *o* parallel to the memb
be considered as made up of straight
drawn parallel to an imaginary straigh
is no load over the center of the two p
are no stresses in the truss-members

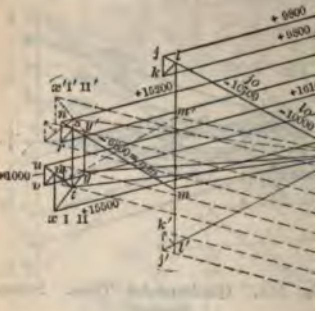


Fig. 36A. Quadrangular Truss. Stress-

economical for very great spans. In t
example, the top chord is curved and
All the other members are in tension.
weight of the roof itself, the diagona
shown in Fig. 37 are all that are need
sure on one side of the roof only, it
diagonals shown by the dotted li

ing the additional set, are not stressed when there is a vertical load ey are omitted in drawing the stress-diagram. To draw the stress- e loads are laid off on a vertical line, as in all the previous examples, being half-way between *e* and *e′* (Fig. 37A). *oa* is the supporting it 1. In drawing the stresses at the different joints, those at joint 1 wn and then those at joints 2, 3, 4, 5, etc., in the order in which they ed (Fig. 37). In the stress diagram, *oa*, equal to the supporting

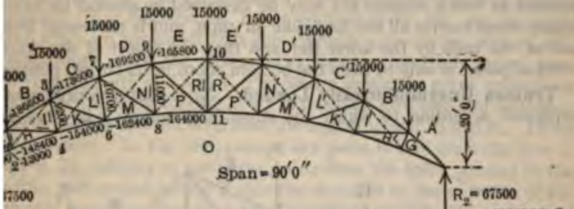


Fig. 37. Bowstring Truss. Truss-diagram

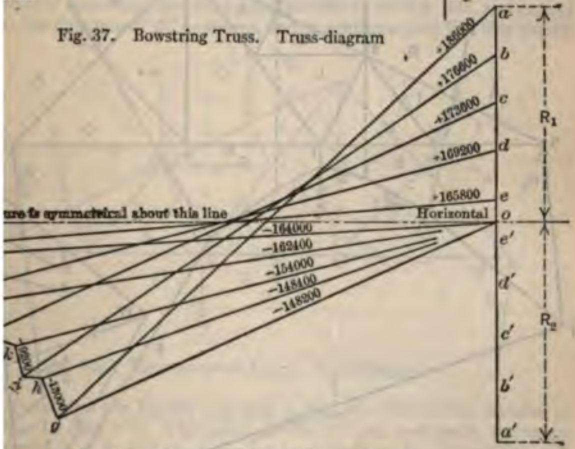
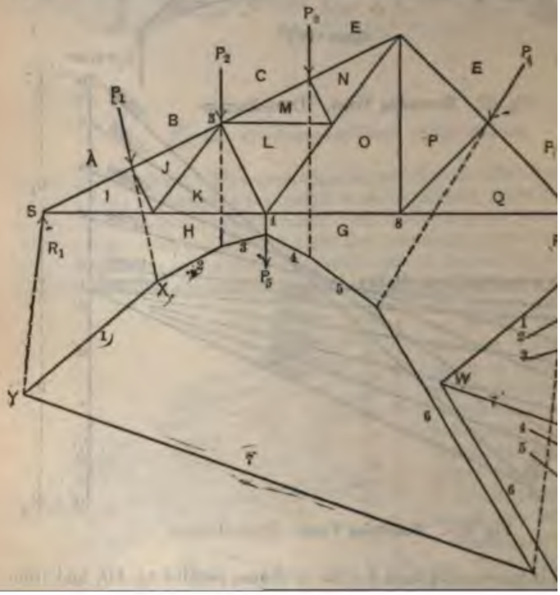


Fig. 37A. Bowstring Truss. Stress-diagram

it 1, is known and from *a* a line is drawn parallel to *AG*, and from rallel to *GO*. These two lines intersect at *g*. The lines repre- stresses in the curved members of the truss are drawn parallel lines connecting the two ends of each curved piece. Thus *ag* is lel to 1–3 and *og* parallel to 1–2. At joint 2, *og* is known, *gh* is drawn *GH* and *ho* parallel to *HO*. At joint 3, *hg* and *ga* have been drawn, is known and *bi* and *ih* are drawn. At joint 4, *oh* and *hi* have been *ik* and *ko* are next drawn to close the polygon. The stress-lines for 8 are drawn in a similar way, and those for 5, 7 and 9 similarly to at 3. After drawing the stress-lines for joint 9, joint 10 is next con- d after the stress-lines *for that joint are determined* the stresses in *bers of the truss are known. The stresses in this particular* example *pounds on the respective lines in the stress-diagram.* It will be

Trusses Unsymmetrically Loaded. Now that the princi
. explained according to which the stress-diagrams may be dra



F THE REACTIONS. If the truss remains in position it follows that
s acting upon the truss, such as the LOADS and REACTIONS, must be
IUM; also since by definition a TRUSS must act as a BEAM, the truss
laced by a BEAM in considering the OUTSIDE FORCES. In Fig. 38,
lines representing the direction of the forces, as shown, until they
ST and assume *ST* to be a simple beam loaded with the forces *AB*,
Beginning with *AB*, to some convenient scale lay off the forces in
wn in Fig. 38A. The broken line *VU* represents the forces in magni-
irection. For equilibrium, forces equivalent to *UV* are required.
lent when we remember that the algebraic sum of the vertical and
components of all the forces acting must respectively equal zero.
hat the supports at *S* and *T*, Fig. 38 are similar in every respect we
that the reactions R_1 and R_2 act in the same direction and that they
to *UV*. This does not determine the magnitudes of R_1 and R_2. These
d as follows: In Fig. 38A, assume any point *W* and draw the lines 1,
n Fig. 38, starting at any point on R_1 draw the line 1 parallel to the
. 38A, and extend it until it cuts the direction or line of action of the
shown; from this point draw a line parallel to 2 in Fig. 38A, and extend
ts the direction of *BC* as shown, and so continue until line 6 is drawn

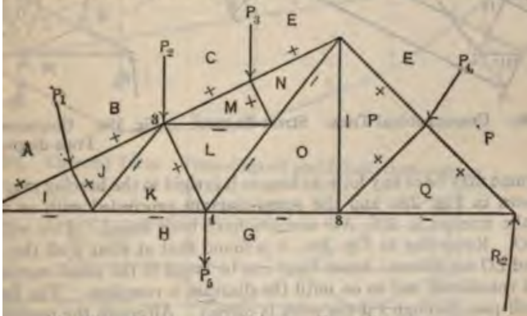


Fig. 38B. Unsymmetrical Truss. Truss-diagram

Draw line 7 in Fig. 38 and then in Fig. 38A draw a line parallel to
until it cuts *UV*. This point divides *UV* into two parts, the upper
agnitude of R_1 and the lower the magnitude of R_2. No trouble will
ced in applying the above method if the following rule is obeyed to
In Fig. 38, the parallels to any three lines in Fig. 38A which form a
st meet in a point. For example, in Fig. 38A lines 1, 2, and P_1,
a triangle, and in Fig. 38, their parallels meet in the point *X*. In
it is not necessary that the forces *AB*, *BC*, etc., be used in order in
t considering them in order, on a simple beam, makes the graphical
in Fig. 38 less complex and avoids many chances of error. The
lined above is general and can be used for forces acting in any direc-
e forces are parallel then the load-line *ab*, *bc*, *ce*, etc., in Fig. 38A, and
will coincide; but the method of procedure remains unchanged.
ll the FORCES acting upon the truss have been determined, for con-
ey will be shown in character, in Fig. 38B, and the STRESSES in the
mposing the *truss will be found. First lay off the forces in exact*
that they form a CLOSED FIGURE, which must be the case if they are
. *The lines with arrow-heads in Fig. 38C show this construction,*

which checks the values of R_1 and R_2 obtained above. This figure rem
same regardless of the interior arrangement of the truss. The construc
the stress-diagram follows the methods given in the previous exampl
point 3 is reached. Here there are three UNKNOWNS, CM, ML and LK
cannot be assumed that ML is the same as JK, as was done in example
11. Let the truss be cut as shown in Fig. 38D, and the actual stresses i
pieces assumed to act against the cut ends, then the frame shown in
and the forces R_1, AB, BC, CE, EN, NO, OG and GH will be in eq

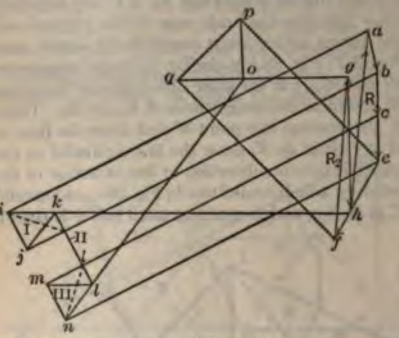


Fig. 38c. Unsymmetrical Truss. Stress-diagram

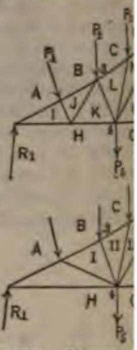


Fig. 38D. Unsymme
Truss-diagra

The frame may be of any form as long as it is rigid so the bracing may
as shown in Fig. 38D and the stress-diagram proceeded with as i
until the stresses in EN, NO and OG have been found. This wil
point O. Returning to Fig. 38B, it is found that at joint 4 all the
KL and LO are known; hence these can be found in the usual mann
is next considered and so on until the diagram is complete. The li
38C will pass through f if the work is correct. Although the meth
mining the CHARACTER OF THE STRESSES has been explained, it will
here in a more general manner. Take, for example, joint 8, in Fig.

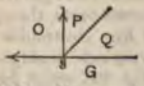


Fig. 38E. Unsymmetrical Truss. Forces at Joint 8

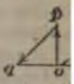
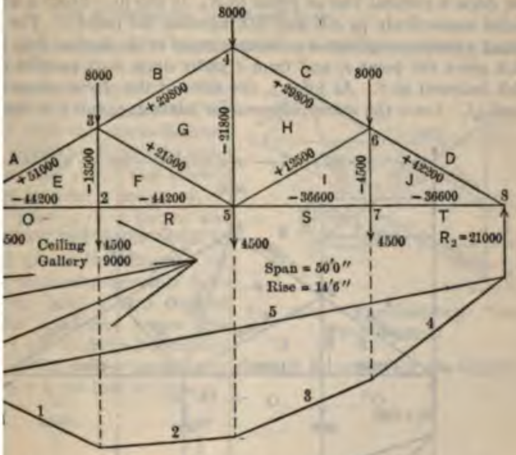


Fig. 38F. U
Truss. Str

is in equilibrium under the action of the stresses go, op, pq and qg, as indicated in Fig. 38E. The stress-diagram for this joint is shown in Fig. 38F, separated from Fig. 38C. It is
assumed that the stress in GO is tension. Then in Fig. 38F, starting
off go, op, pq and qg, placing the arrow-heads as shown. Transf
arrow-heads to the ends of the cut pieces in Fig. 38E indicates
KIND OF STRESS. The following examples illustrate the above met

Unsymmetrically-loaded Truss. Example 24. Fig. 39 repres
gram of a truss similar to that shown in Fig. 1, but of a greater spar
a gallery supported from it at one side only. The approximate roof
loads are indicated by the figures near the arrows, and the weigh
one truss from the gallery would be about 9 000 lb. The first
drawing the stress-diagram is to determine the reactions at the

give the supporting forces. This is readily done in this example OF MOMENTS explained on pages 322 to 324. Moments are first nt 1. As the loads at joints 2 and 3 have the same arm, they her before multiplying by the arm. The loads at joints 4 and 5



King-rod Truss. Truss-diagram and Equilibrium-polygon

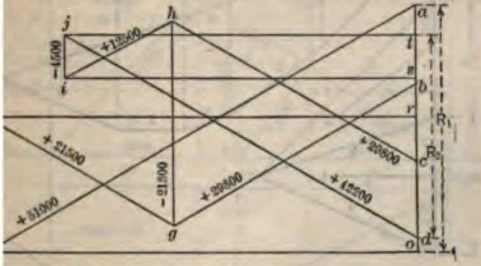


Fig. 39A. King-rod Truss. Stress-diagram

and 7 are treated the same way. The moments about joint 1 will

+ 4 500 + 9 000) = 21 500] lb × 12½ ft = 268 750 ft-lb
+ 4 500) = 12 500] lb × 25 ft = 312 500 ft-lb
+ 4 500) = 12 500] lb × 37½ ft = 468 750 ft-lb

he sum of the moments = 1 050 000 ft-lb

se CLOCKWISE MOMENTS about joint 1 *must be balanced by the* *ISE MOMENT of R_2, the LEVER-ARM of which, with reference to* *Knowing the arm, 50 ft, the force R_2 is obtained by dividing* *oments of the loads by the span. Dividing 1 050 000 ft-lb by*

50 ft, the result is 21 000 lb, which is the reaction or supporting force at j and R_1 must equal the difference between the sum of the loads and R_2 sum of the loads is 46 500 lb and subtracting from this 21 000 lb, the rem 25 500 lb, is the value of R_1. The stress-diagram (Fig. 39A) may now be First draw a vertical line *oa* equal to R_1, 25 500 lb. From *a* and *o* dra parallel respectively to *AE* and *EO*, locating the point *e*. For the stre at joint 2 measure up from *o* a distance equal to the load at that joint, which gives the point *r*, and from *e* and *r* draw lines parallel to *EF* which intersect at *f*. At joint 3, the sides of the stress-polygon are *fe*, *bg* and *gf*. Draw the stress-polygons for joints 4, 5, and 6 in the order

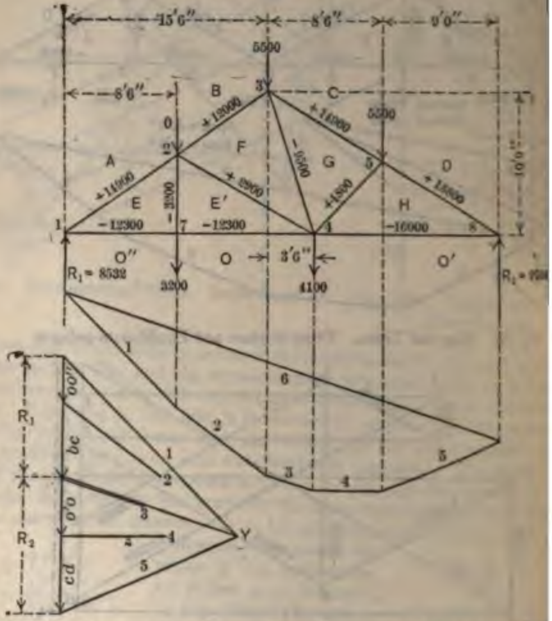


Fig. 40. Unsymmetrical Truss. Truss-diagram and Equilibrium-polyg

they are numbered. At joint 6 the sides of the stress-polygon are *ih*, and *ji*. If the diagram has been correctly drawn, the line *ij* will be to the load at joint 7. The sides of the stress-polygon for joint 7 are 4 500 lb, *si*, *ij* and *jt*, the only line to be drawn being *jt*, which must be to *JT*. Consequently *j* must be exactly opposite *t*, or the polygon will The distance *dt* should be equal to R_2.

Unsymmetrically-loaded Truss. Example 25. Fig. 40 is the di a wooden roof-truss. The actual loads were about as given on the There were purlins at joints 3 and 5 only, and the ceiling below was by rods from joints 4 and 7, joint 4 being fixed by the framing of th The moments of the loads about joint 1 are:

3 200 lb × 8½ ft = 27 200 ft-lb
5 500 lb × 15½ ft = 85 250 ft-lb
4 100 lb × 19 ft = 77 900 ft-lb
5 500 lb × 24 ft = 132 000 ft-lb

Sum of moments = 322 350 ft-lb

e sum of the moments by the distance between the supporting forces,
s 9 768 lb as the value of R_2.
f the loads is 18 300 lb. Sub-
768 lb, 8 532 remains as the
. To draw the stress-diagram,
o''a equal to 8 532 lb equal
lraw *ae* and *eo''*. The sides of
olygon for joint 2 are *ea*, *ab*, *bf*,
At joint 3, *fb* is known and
ured down and made equal to
g and *gf* are then drawn. At
rt by measuring upwards from
locating point *o'* and draw *gh*
At joint 5, *hg* and *gc* are now
quals 5 500 lb and a line from *d*
rallel to *DH*. This should pass through *h*, completing the diagram.
n rod 2–7 is the load at joint 7.

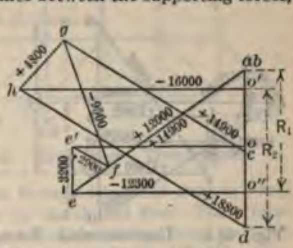
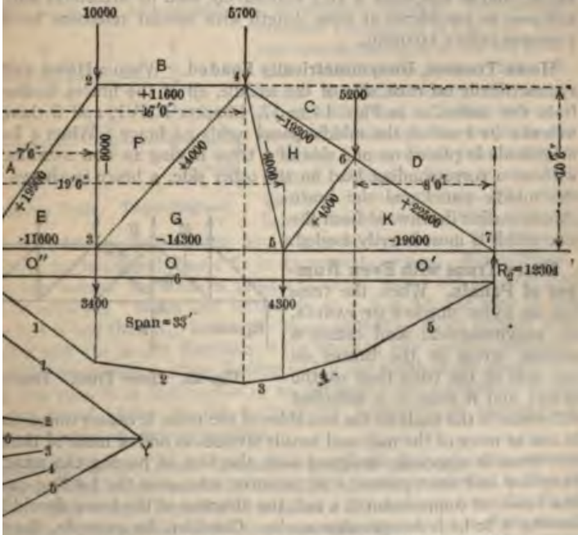


Fig. 40A. Unsymmetrical Truss. Stress-diagram



1. Unsymmetrical Truss. Truss-diagram and Equilibrium-polygon

etrically-loaded Truss. *Example 26. Fig. 41 is the truss-diagram truss in the same building in which the truss shown in Fig. 40 was g moments about joint 1, there results, for the sum of the moments,*

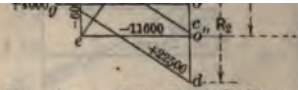


Fig. 41A. Unsymmetrical Truss. Stress-diagram

40A and 41A, are unsy are of the same genera the stresses are all of t when the supporting f This condition holds tru gular trusses, but for t zontal or curved chord loading usually causes a REVERSAL OF THE STRESS IN KIND in o diagonals or verticals; and if the truss contains any four-sided tional diagonal is generally required. This is particularly t TRUSS; and as this truss is very extensively used by architect will now be considered at some length with special reference UNSYMMETRICAL LOADING.

Howe Trusses, Unsymmetrically Loaded. When a How symmetrically on each side of the middle, all of the braces i from the center, as in Figs. 14 to 17, Chapter XXVI; and i NUMBER OF PANELS, the middle panel needs no brace. Whe magnitude is placed on one side of a truss having an ODD NU without a corresponding load on the other side, a brace is al the middle panel and the brace should incline downward from the side which is most heavily loaded.

Howe Truss with Even Number of Panels. When the truss has an EVEN NUMBER OF PANELS, an unsymmetrical load causes a greater stress in the braces on one side of the truss than on the other; and if there is a sufficient difference in the loads on the two sides of the truss, it causes co

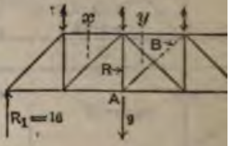


Fig. 42. Howe Truss.

he DIRECTION OF THE BRACE IS REVERSED, as shown by the full line.
compression and the vertical *R* has no stress except that caused by
ad of 9 tons. There are the same results when the load of 9 tons
the joint directly above, instead of at the lower joint, although in
re is no stress at all in *R* except that due to the weight of the tie-beam.
ad of 9 tons is reduced to 6 tons, no brace is required in the third
when the bottom load
6 tons, a brace in the
ction is required, as
. 43. (See page 1006.)

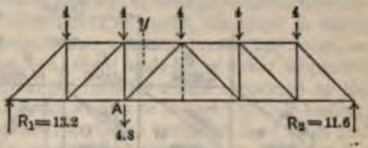
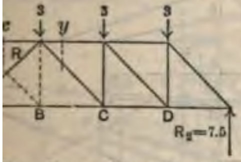


Fig. 43. Howe Truss. Truss-diagram

**russ with Uneven
Panels.** In the five-
shown in Fig. 44, a
tons at *A* requires
ment of braces shown
lines, and when the load at *A* is increased to more than 15 tons,
needs to be reversed, as shown by the dotted line. The stress-
ays shows in which direction any brace should be placed to be in
; but this may be determined also by the following rule. When the
oads to the left of any section, taken between R_1 and the middle, is
the reaction R_1, the direction of the brace cut by that section must
from its normal direction. When the sum of the loads is less than
should be in its normal position. When the sum of the loads, to
he section, is just equal to R_1, no brace is required. For example,
consider a section at *x*, Fig. 44. Here
the sum of the loads to the left is 10.5
tons, which sum is less than R_1;
hence the brace should be in its
normal direction. If the section is
at *y*, the sum of the loads to the
left is 13.5 tons, which sum is greater
than R_1; hence a brace is required,
slanting downward from the more
heavily loaded side. When the sec-
Fig. 42, the load to the left is 4 tons, an amount less than R_1;
ace in that panel should be in its normal position. When the sec-
the sum of the loads is greater than R_1; hence the brace in that
be reversed. When the section is at *y*, Fig. 43, the sum of the
left is less than R_1; hence the brace should be in its normal posi-
is rule the proper direction of the brace in any panel is indicated,
the complication of the loading and of the width of the panels; but
the rule, it is first necessary to determine the supporting forces,
be found either by the METHOD OF MOMENTS, as explained in
or by the GRAPHICAL METHOD.



Iowe Truss. Truss-diagram

etrical Howe Truss. Example 27. As an example of an unsym-
WE TRUSS unsymmetrically loaded, the truss represented in Fig. 45
. This truss is supposed to support a flat roof and a wooden tower
own. The position of the tower necessitates a division of the panels
so that the truss is quite unsymmetrical. It is assumed that the
roof, snow, and tower constitute the loads in pounds at the upper
ted by the figures. The graphical determination of the reactions
s clearly shown in Figs. 45 and 45A. The only panels of this truss
is any question as to the direction of the braces are the third and

fourth. Taking a section at x, the sum of the loads to the left R_1; hence the brace should be placed as drawn. A section t makes the sum of the loads to the left less than R_1; and hence t be in its normal position. The stress-diagram of this truss is starting with wa equal to R_1, and going from joint to joint as in ples. The completed stress-diagram is shown in Fig. 45A.

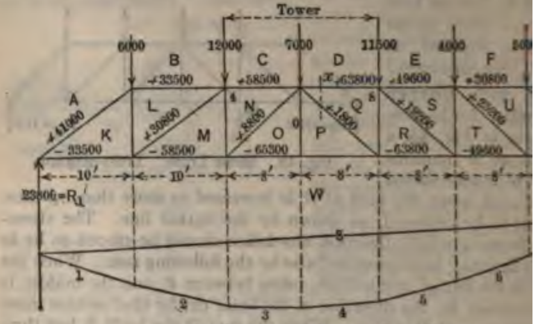


Fig. 45. Howe Truss. Truss-diagram

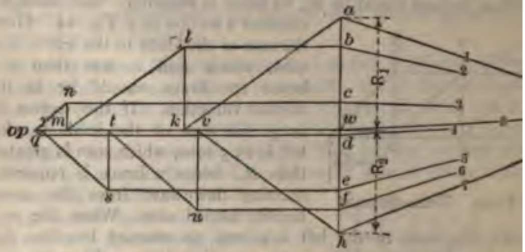


Fig. 45A. Howe Truss. Stress-diagram

Counterbraces. These are EXTRA BRACES that are put stresses are REVERSED IN CHARACTER by a load which may be a and then removed. For illustration, consider the truss represe and 43. Here it has already been shown that when the load at the brace in the third panel should be in the position shown in when the load is greater than 6, the brace should be in the no ıe condition resulting from the REMOVAL OF THE VARIABLE LOAD. ıld be designed to resist both conditions. SNOW is a VARIABLE ı such trusses are often subjected; but as it is nearly uniformly ›r the roof, it does not change the CHARACTER OF THE STRESSES nembers. If a truss, therefore, is designed for a MAXIMUM SNOW- ıre than strong enough when there is no snow. Moreover, the ngth of the chords is usually sufficient to resist any slight inequality . The principal VARIABLE VERTICAL LOADS, therefore, to which ay be subjected and which require counterbraces, are those due of people, merchandise, etc., these loads being either suspended by rods or brought upon the truss by a floor supported by the The truss shown in Fig. 45, also, is an instance of such loading. ven by the figures indicate merely the combined dead loads and During a high WIND the weight on the LEEWARD SIDE of the tower sed and on the WINDWARD SIDE decreased, so that when the wind right, the load at 4 is greater and at 8 less than indicated; while blows from the left the load is increased at 8 and decreased at 4. COUNTERBRACES in both the third and fourth panels. As counter- ıarm, even if never brought into action, it is always well to use ddle panels wherever the loads are at all variable.

Trusses. These trusses may be considered as UNSYMMETRICALLY ɛs, for although the loads may be symmetrical in relation to the usually unsymmetrical in relation to the supports. The method THE SUPPORTING FORCES and drawing the stress-diagrams is shown ıg examples:

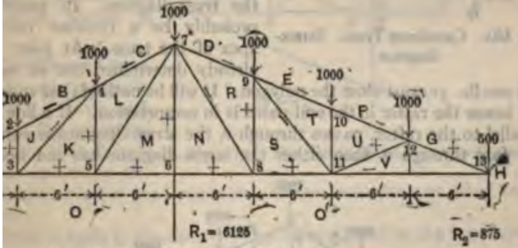


Fig. 46. Cantilever Truss. Truss-diagram

Truss. **Example 28.** Fig. 46 is the diagram of a CANTILEVER

and as the total load is 7 000 lb, R_2 must be 875 lb. The stress-dia
commenced either with the forces at joint 1 or with those at joint
the external loads were laid off from left to right in the preceding e
same order is used here. Comm
with joint 1, lay off on a vertical
oa equal to 500 lb, which acts
draw *ai* and *io* parallel respect
and *IO*. The forces act from *o*
to *i* (from the joint) and from *i*
the joint), showing that *AI* is in
IO in compression, a REVERS
CHARACTER OF THE STRESSES
the corresponding members of
ported at both ends. Next, at
stress *ia* is now known, and *ab* e
lb is laid off; then *bj* and *ji* a
being in tension and *JI* in
The forces at joint 3 are next
then those for the remaining j
order in which they are number
6, the first force known is the sup
R_1, represented by *o′o* laid off e
lb and acting upward. The
polygon of forces for joint 6 a
and *no′*. The stress in *MN* is
supporting force *o′o*, which is
the truss-diagram. In practic
probably be a COLUMN conti
apex of the truss. At joint 12
already determined are *vu*, *uf*,
to 1 000 lb. *gv* must close the polygon. It will be noticed that *gv* acts
12; hence the rafter in the end-panel is in compression. If a line d
parallel to the rafter, passes through *v*, the stress-diagram is corre
not pass through *v*, then either the stress-diagram has not been

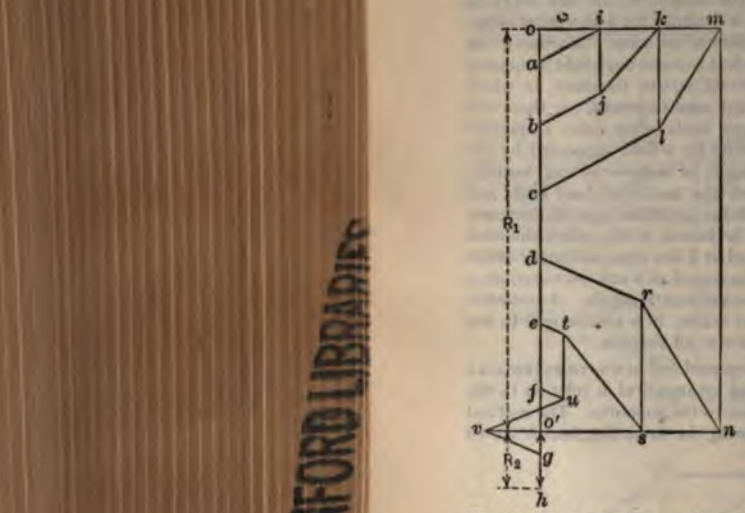


Fig. 46A. Cantilever Truss. Stress-diagram

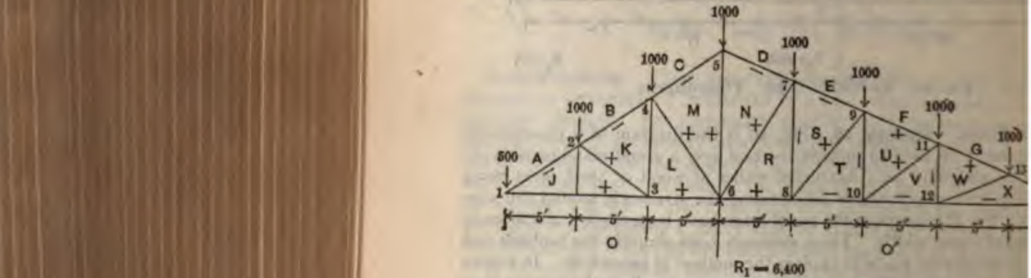


Fig. 47. Cantilever Truss. Truss-diagram

sufficient accuracy or an error has been made in computing the sup
In drawing the stress-diagram for CANTILEVER TRUSSES, it is impo
in mind THE DIRECTION IN WHICH THE FORCES ACT, in order to d
members are in compression and which in tension.

Truss. **Example 29.** Fig. 47 is the diagram of a truss similar ıat shown in Fig. 46, but with the DIAGONAL BRACES INCLINED E DIRECTION, so as to cause them to be in compression and the ısion. The supporting forces are found by the same methods ple 28, and the also, is drawn by sed for Fig. 46A. owever, the stress post *MN* is con- han the reaction large portion of nsmitted to joint s *LM* and *NR*. three sections of he right side are and three sec- ottom chord are is is because in projection of the roportion to the less than it is in the stress-lines of the load-line e STRESSES ARE IND. This truss ted to wooden th vertical rods truss shown in

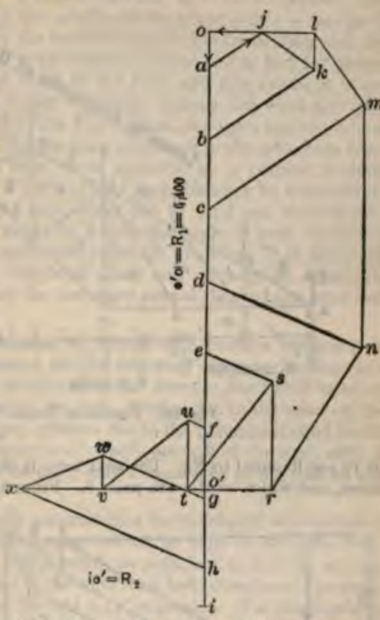


Fig. 47A. Cantilever Truss. Stress-diagram

ıntilever Truss. n this example, vn a truss with ıt the outer end , so that R_1 acts determine the character of the supporting forces moments are taken about s:

ıts of loads to the right of joint 7, the figures on the force-arrows ram indicating thousands of pounds:

8) + (5 × 16) + (5 × 24) + (12.5 × 32) = 640 000 ft-lb

ıts of loads to the left of joint 7:

(2.5 × 24) + (5 × 16) + (5 × 8) = 180 000 ft-lb

ıts act in opposite direction with reference to the center of 6, the smaller sum, is subtracted from the larger, leaving an nent of 640 000 ft-lb − 180 000 ft-lb = 460 000 ft-lb, tending down on the right of R_2 at joint 6 and to lift it up on the left. ıst be resisted by the moment of the reaction R_1, which has an ividing 460 000 ft-lb by 24 ft, 19 250 lb results as the reaction requires a downward *force of this magnitude to maintain the um. As the support at 6 must resist this downward pull as will equal the sum of the loads plus the pull* R_1, or 45 000 lb

+ 19 250 lb = 64 250 lb. Having obtained the value of the supporti
the stress-diagram is drawn by laying off on a vertical line *oa* downwa

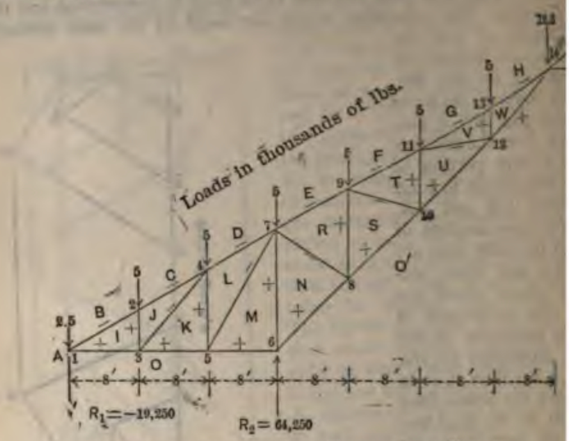


Fig. 48. Cantilever Truss. Truss-diagram

to 19 250 lb equal to R_1. The next force is the load of 2 500 lb, wh
down, and which locates the point *b*. From *b* a line parallel to *BI* i

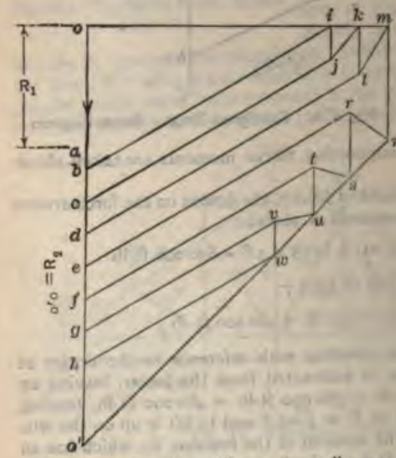


Fig. 48A. Cantilever Truss. Stress-diagram

from *o* a line pa
locating the poin
from joint 1 and
it, showing that
tension and *IO* in
The remainder o
diagram is drawn
methods employ
diagrams of Fig
47A. At joint
polygon is begu
force R_2 or *o'o*,
upward, and th
of which must be
sequently *o'* is
measuring down
64 250 lb. The
stress-polygon f
are *o'o*, *om*,
After *gh*, the loa
is laid off, the
tance *ho'* should
to the load at
12 500 lb. If *R*

been correctly computed and the stress-diagram accurately dr
s, *u* and *w* will fall in the line *no'*.

6. Determination of Wind-Load Stresses

ads. Thus far the stresses due to VERTICAL LOADS only have been
:he pressure of the WIND being combined with the DEAD LOAD and
is acting vertically. For TRIANGULAR and FINK TRUSSES this
ifficiently accurate, as the wind-pressure never causes a maximum
ess of that obtained by the method explained in connection with the
amples. For TRUSSES WITH CURVED CHORDS and in fact for almost
STEEL TRUSSES except those of the FINK and FAN TYPES, it is not
ider wind-pressure as acting vertically, because the wind acts gen-
irection at right-angles to the roof-surface, and upon but one side of
given time, thus loading the truss unsymmetrically and often caus-
of an opposite kind from those produced by a vertical loading.
h are inactive under a vertical load may therefore be necessary to
rce of the wind, or the total stress due to wind and vertical load
ay be greater than it would be if the wind-pressure were considered
l load. To design a roof-truss correctly, therefore, it is necessary
e the stresses due to VERTICAL LOADS and WIND-LOADS separately
nbine them so as to get the GREATEST STRESS that may be produced
robable conditions.

Chords. In the calculation of trusses with CURVED CHORDS it is
ractice to find the stresses for the following different loadings and
e them to obtain the maximum stress: Stresses due to the wind on
:he truss nearer the expansion-end; stresses due to the wind on the
russ nearer the fixed end; stresses due to the permanent dead loads;
to snow covering the entire roof or only one-half of the roof; and, in
, stresses due to snow covering only a small area of the roof on one

d Snow. It is generally assumed that the maximum wind-pressure
w-load can not act on the same half of the truss at the same time.
vith straight rafters it will generally be sufficient to find the stresses
ermanent dead load, and to the wind from both directions, disregard-
-load when the pitch of the roof is 45° or greater. For the Northern
the pitch is less than 30°, it is well to consider that a heavy sleet
oth sides of the roof at the time of a heavy wind and to add about
ft of roof-surface to the dead load to allow for it. In localities where
alls may be expected, the stresses due to the full snow-load should
d, as these combined with the permanent dead load may exceed
dead load, sleet and wind-pressure.

ess-Diagrams. These are affected by the manner in which the truss
. If both ends of the truss are fixed, the WIND-REACTIONS are paral-
sultant wind-load; if one end is free to move horizontally, that is,
or supported on a ROCKER, the reaction at the roller-end is vertical
the fixed end inclined. "If one end be fixed and the other merely
pon a smooth IRON PLATE, the reaction at the free end may have a
omponent equal to the vertical component multiplied by the COEFFI-
ICTION, which is about one-third."

d Free Ends of Trusses. Wooden trusses may be considered as
ends. STEEL TRUSSES, when supported on masonry walls, should
d FIXED and the other FREE to move; and when the span exceeds
e end should be *supported on ROLLERS to permit of* expansion or
When steel trusses are supported by steel columns, as in steel mill-
trusses are RIGIDLY ATTACHED to the columns and no provision

is made for expansion. In such buildings the wind-pressure causes a STRESS in the columns, which must be provided for.

Truss with Fixed Ends. **Example 31.** Wind-pressure is usually assumed to be applied uniformly over one side of the roof and to act at right-angles to the surface of the roof. The joint-loads or panel-loads, therefore, are proportional to the roof-areas supported. When the joints divide the rafter into panels of equal length, the joint-loads are uniform, except for the joints at the edges of the The actual wind-pressure is obtained by multiplying the roof-surface by the values given in Table IX, page 1053. For this example the triangular truss in outline by Fig. 49 is considered and it is assumed that the span and spacing of the truss are such as will give a load of 1 000 lb at joints 2 and 4. The at joints 1 and 5 are only one-half of those at 2 or 4. To find the supporting forces or reactions, draw a line representing the resultant of the loads, the bottom chord at X. As the loads are symmetrical the resultant acts at the middle of the rafter and at right-angles to it. The reactions R_1 and inversely proportional to the two segments into which a horizontal line the points of support is divided by the resultant, or in this case to X-7 and

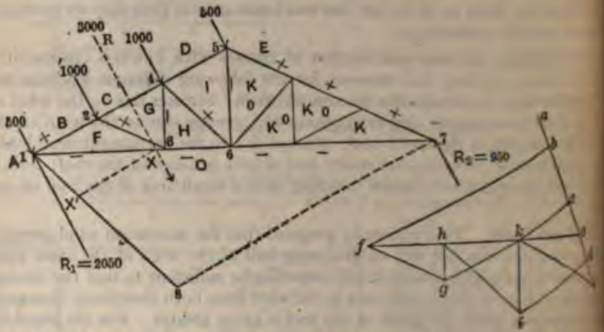


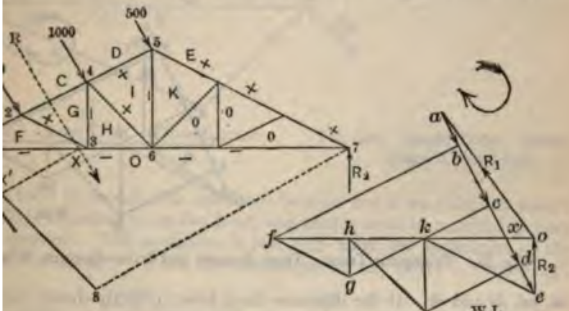
Fig. 49. Triangular Truss. Truss-diagram

Fig. 49A. Triangular Truss. Stress-diagram

the greater reaction being at joint 1. The sum of the reactions are equal to the sum of the loads. To find the reactions graphically, draw a line from joint at any angle, say from 30° to 45°, and measure off a distance equal to the load. In Fig. 49 the line 1–8 represents 3 000 lb. Join 7 and 8, and from draw a line parallel to 7–8, intersecting 1–8 at X'. Then 8–X' is the reaction at joint 1 and X'–1 the reaction at joint 7. To draw the stress-diagram, Fig. first draw the load-line *ae* equal to the sum of the loads, in this case 3 000 lb, perpendicular to the rafter 1–5, and divide it so that *ao* is equal to X'-8. Then at joint 1, *oa* is the supporting force, *ab* is 500 lb and *bf* and *fo* are drawn parallel respectively to BF and FO, intersecting at *f*. The external forces and act in the direction *oa*, *ab*, *bf* and *fo*, showing that BF is in compression and in tension. At joint 2 the stress-lines are *fb*, *bc* equal to 1 000 lb, *cg* and *gf*. The stress-lines at joint 3 are *of*, *fg*, *gh* and *ho*; at joint 4, *hg*, *gc*, *cd*, *di* and *ih*; and joint 5, *id*, *de*, *ek* and *ki*. If the load-line has been correctly divided at the stress-lines have been drawn exactly parallel to the lines of the truss, the *k* will fall vertically above the point *i*. At joint 6 the stress-lines are *ok*, *ki*, *ko*. As the figure must close by a horizontal line through *o*, it is evident

he truss-diagram cannot be represented, and therefore there can be
his member when the wind is from the left. At joint 7 the reaction
acting up, and *ok* and *ke* must close the figure, showing that the line
the stress in the entire length of the right rafter, and that there is
the bracing on that side of the truss when the wind is from the left.
ver, either the lower chord or the rafter is not straight, some of the
at side come into action. By noting the character of the stresses in
is seen that the different members of the truss have the same kind
is produced by vertical loads. As the wind may blow from either
is evident that both sides of the truss must be made alike. This
strates the method of drawing the stress-diagram for any truss with
fter when both ends of the truss are fixed.

Rollers. Example 32. When one end of the truss is FREE TO
eaction at that end must always be practically vertical, and this
ves a considerable variation of stress when the wind is on different
roof; so that it is necessary to draw two wind-stress diagrams, one



Triangular Truss. Truss-diagram and Stress-diagram, Wind Left

OM THE LEFT, marked W.L, and one for WIND FROM THE RIGHT,
. It is customary with authors when writing on this subject to
t the ROLLERS are always under the right-hand support, and this
llowed here. In practice the ROLLERS may be placed under either
sides of the truss are usually proportioned to the maximum stresses.
nple we will take the same truss-diagram that was used in Fig. 49,
t again in Fig. 50, which is drawn to show WIND FROM THE LEFT.
oad-line 1–8 and divide it at X', as in example 31. Draw a line *ae*,
r to the rafter and equal to 1–8 in length, and divide it into two
the same proportions. Through x' on *ae* draw a horizontal line,
e a vertical line, the two intersecting at *o*. Then *eo* represents the
tion at joint 7 and *oa* the reaction at joint 1. The stress-lines at
a, *ab* equal to 500 lb, *bf* and *fo*. At joint 2: *fb*, *bc*, *cg* and *gf*. The
the diagram W.L. is completed exactly as described for Fig. 49A,
erence between the two being the location of point *o*, which gives
esses in the bottom chord for the truss of Fig. 50. Fig. 51 represents
ss with *WIND FROM THE RIGHT. To draw the stress-diagram* W.R.
perpendicular to the rafter and equal to the total load, 3 000 lb,
at x' into two segments of the same proportions as the segments

of the line 1–8, Fig. 50, the longer segment being at the top. To
tions draw a horizontal line through x' and a vertical line through
intersecting at o. Then do is the reaction at joint 1, and ol the
10. For this diagram it is better to start with joint 10 and tak
the reverse order from that in which they were taken before. T
at joint 10 are ot, ts equal to 500 lb, sn and no; at joint 9, ns,
at joint 8: on, nm, ml and lo; at joint 7, lm, mr, re, ek and kl;

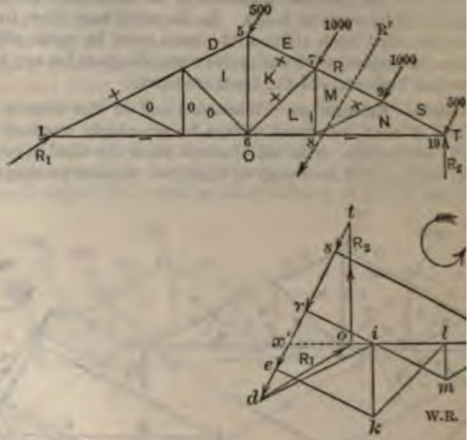
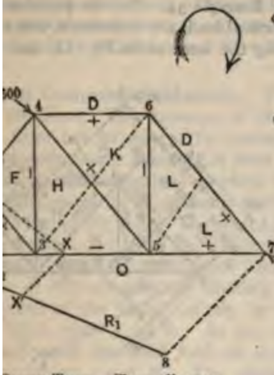


Fig. 51. Triangular Truss. Truss-diagram and Stress-diagram,

ke, ed, di and ik. If the diagrams have been correctly drawn
fall vertically above the point k. On comparing the two diag
and W.R. it is seen that the stress-lines for the rafters and braces
length and that the stresses are of the same character in both, bu
in the bottom chord is considerably less when the wind is from
condition does not apply to all trusses, however, so that it is b
two stress-diagrams for wind from both directions.

Queen Truss. **Example 33.** Fig. 52 represents the outline
TRUSS for a roof having a rise of 14½ in in 12 in. As the truss
supports are considered fixed. Joint 2 divides the rafter into
consequently the wind-load at this joint is twice that at join
convenience it is assumed that the wind-load at joint 2 is 1 000 lb
and 4, 500 lb. The resultant is 2 000 lb acting through joint
the tie-beam at X. To find the supporting forces, draw the li
2 000 lb and connect 7 and 8. From X draw a line parallel to
1–8 at X'. Then 8–X' is R_1 or the supporting force at joint 1
the supporting force at joint 7. Begin the stress-diagram (Fig.
the line ad at right-angles to the rafter 1–4, and equal in length to
By means of dividers locate the point o so that oa equals 8–X'.
lines for joint 1 are oa, ab, be and eo; at joint 2, eb, bc, cf and fe
ef, fh and ho; and at joint 4, hf, fc, cd, dk and kh. It is seen that
gon at joint 4 will not close without the brace KH, because th
drawing the polygon is at h, and a horizontal line throug

ueen-rod truss, therefore, requires braces in the middle panel l-stress. With the wind from the right, a brace is required from . At joint 5 the stress-lines are *oh*, *hk*, *kl* and *lo*. It should be cts towards the joint, showing that *LO* is in compression. At



Queen Truss. Truss-diagram

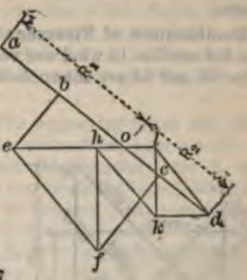
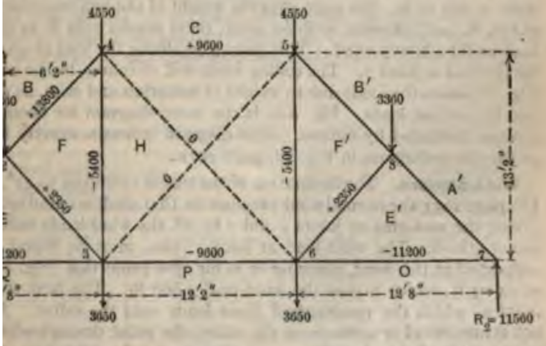


Fig. 52A. Queen Truss. Stress-diagram, Wind Left

em as though this could not be true, but if we glance at joint 7 thrusting in on the joint, and that a strut is required to keep tion. This condition is true only when the inclination of the than 45°. When the inclination of the rafter is exactly 45°,



Truss. Truss-diagram. (See, also, Figs. 3, 12 and 54 and Chapter XXVIII, Fig. 1)

s in *LO*, and when the inclination is less than 45°, *LO* is in tens-lines for *joint 6 are lk, kd and dl. If no errors are made*, a *rallel to DL passes through the point l*, previously obtained. A *racy in locating the point X′*, or in drawing the stress-diagram, however, causes the line through *d* to pass to one side or the other of and if this happens, it shows that there has been some inaccuracy In practice, a slight divergence does not materially affect the stress. the sides of the stress-polygon are *ol*, *ld* and *do* = R_2, the lines being drawn.

Combination of Stresses. **Example 34.** For the purpose of the stresses due to wind and vertical loads are COMBINED, the truss- Figs. 53 and 54 are shown, being the same as in Fig. 12, and

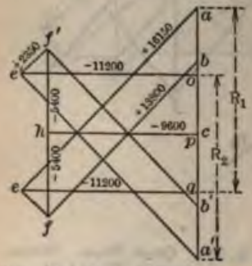


Fig. 53A. Queen Truss. Stress-diagram

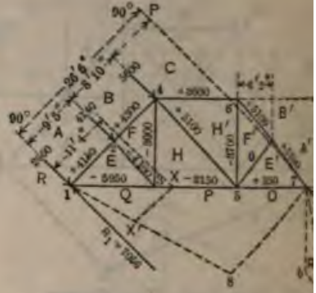


Fig. 54. Queen Truss. Truss-diagram. Figs. 3, 12 and 53 and Chapter XXVI

truss shown in Fig. 3. The stresses first determined are those due to of the roof and ceiling and to an allowance of 10 lb per sq ft for page 1055 the roof-area supported at joint 2 was found to be $147\frac{1}{2}$ joint 3, 200 sq ft. On page 1055 the weight of the roof was estimated per sq ft, and allowing 10 lb for sleet, there results $22\frac{3}{4}$ lb as the load under a heavy wind. This gives 3 360 lb for the load at joint 2 for the load at joint 3. The ceiling-loads will, of course, be the same as Fig. 53 shows the loads due to weight of materials and sleet, as and the ceiling-loads. Fig. 53A is the stress-diagram for these loads, stresses indicated by figures. This diagram is drawn exactly in the as the stress-diagram in Fig. 12, page 1071..

Wind-Stresses. The inclination of the roof is very close to 45°, and IX, page 1053, the normal wind-pressure for that angle is found to be 28 plying the roof-area at joints 2 and 3 by 28, the wind-loads indicated are obtained. The wind-load at joint 1, also, must be found. The supported at this joint, allowing 17 in for eave-projection (Fig. 3) is or 95 sq ft, which makes the wind-load 2 660 lb. The next step is point at which the resultant of these loads cuts the rafter. As the not symmetrical or uniform on the rafter, the point through which the acts must be determined by means of moments about joint 1. The loads at joints 2 and 4 are figured on the truss-diagram (Fig. 54). The are

$$4\,140 \text{ lb} \times 9\tfrac{5}{12} \text{ ft} = 38\,985 \text{ ft-lb}$$
$$5\,600 \text{ lb} \times 18\tfrac{1}{4} \text{ ft} = \underline{102\,200} \text{ ft-lb}$$
$$\text{The sum of the moments} = 141\,185 \text{ ft-lb}$$

e sum of all the loads, or 12 400 lb, and the distance of its point
ı 1 is found by dividing the sum of the moments by the result-
35 ft-lb divided by 12 400 lb = 11.4 ft. Measuring off 11.4 ft
joint 1 and drawing a line at right-angles to it intersecting the
ıt X is determined. From 1 the line 1–8 is drawn at any
length to the sum of the loads, 12 400 lb, and 7–8 is drawn.
lrawn parallel to 7–8, intersecting 1–8 at X'. Then 8-X'-is
ng force at joint 1 and X'–1 is R_2 or the supporting force at

ѕs Computed by Moments. The supporting forces may also
ıoments. The moments of the loads about joint 1 tend to
rom left to right. To prevent this rotation there is the
porting force R_2 acting at joint 7 to rotate the truss from right
ain equilibrium, the moment of R_2 about joint 1 must just
the moments of the loads about the same point. This sum
be 141 185 ft-lb. The arm of R_2 is the perpendicular distance
action and joint 1. Continuing R_2 above the truss until it
ed line at P, the distance from 1 to P is 26.5 ft. Knowing
of R_2 is obtained by dividing the
nts of the loads, 141 185, by the
This gives 5 344 lb. As the sum
t equal the total load, R_1 equals
lb, or 7 056 lb. The distance
uld scale reasonably close to these
the supporting forces, the stress-
is drawn exactly as described for
inclination of the rafters is a
45°, OE' is in compression, but
small. The figures on Fig. 54A
es in pounds. The stresses may
and should be arranged as in the following table. In
d-stresses, it should be remembered that the wind may blow
of the truss, and the greatest stress liable to occur should be

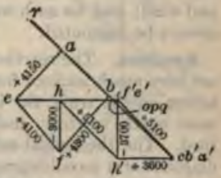


Fig. 54A. Queen Truss. Stress-diagram, Wind Left

Stresses for the Trusses Shown in Figs. 12, 53 and 54

)ead weights and sleet	Wind-stresses	Totals	Stresses (Fig. 12)

greater than in *FH*; hence the stress in *H'F'* should be tabul
in *OE'* slightly reduces the tension due to the dead load, but
increases it, the stresses in *EQ* and *HP* should be tabulated.
truss should of course be made alike, and two braces should
middle panel. In the fifth column of the table are given the
ceiling-load and a vertical load on the roof of 42¾ lb per sq ft
the stress-diagram, Fig. 12. Comparing the stresses in th
columns, it is seen that except for the brace *EF*, and for the tw
obtained by combining snow and wind and adding to the dead
than the totals due to wind, dead weight and sleet. Vertic
cause no stresses in the braces of the middle panel, and unles
are drawn, it is necessary to estimate the sizes of these brace
these braces, however, are so small that large pieces of timbe
The stresses given in the fourth column are unquestionabl
real stresses are likely to be than those in the fifth column. I
in a warm climate where there is no sleet, these stresses may
by omitting the 10 lb per sq ft added for sleet. If, on the
clination of the roof is less than 30°, the stresses produced
snow without wind generally exceed the sum of those due to
and wind; and for such roofs the stresses due to the maximu
always be computed.

Reactions. The reactions, or supporting forces of the trus
are very much inclined from the vertical. As the dead load,
acting on the truss, the inclination of the real reaction is
more nearly vertical; and when there is no wind the reactio
tical. The theoretical reaction, due to both wind-load and
diagonal of a parallelogram, the two adjacent sides of whic
for the dead load and wind-load drawn to the same scale. T
represents the reaction due to the wind and *b*–7 the vertical
dead load and drawn to the same scale, then R'_1 is the resultan
somewhat, however, by friction. Examples 31, 32 and 33
general method of drawing WIND STRESS-DIAGRAMS, and are
the student to draw those diagrams for most trusses with str
trusses with curved rafters the diagrams become more co
reader is referred to Graphical Analysis of Roof Trusses, by
and to other standard handbooks on the subject.

7. Trusses with Knee-Braces

Knee-Braces are generally used to give greater stability t
whole when roof-trusses are supported by COLUMNS. U
vertical loads the stresses in these members are usually assu

nidway between the bottom of the knee-brace and the masonry
iivalent to assuming a PIN at this point), and so that the top attach-
hose of the knee-braces may be considered as PIN-CONNECTIONS.
russ and loading shown in Fig. 55, it is clear that the outside forces
equilibrium, and, unless the points M and N are unlike in some

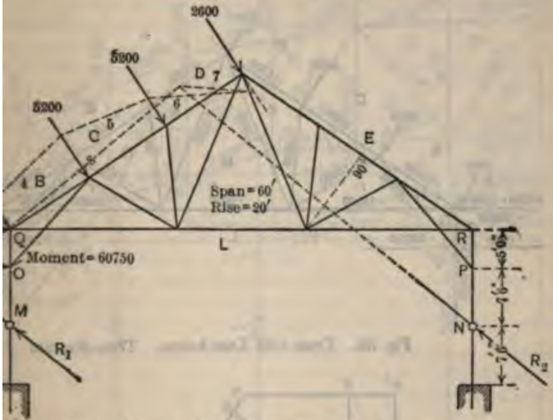


Fig. 55. Truss with Knee-braces. Truss-diagram

he reactions at these points will be parallel to the direction of the
the wind-forces. Lay off to any convenient scale the wind-forces
hown in Fig. 55A. Then XY is the direction and magnitude of the
nd-pressure and also the direction of R_1 and R_2. The magnitudes
are found by means of the equilibrium polygon explained on page 1097.
to SX and R_2 to YS. These
correct in direction and mag-
s some condition is imposed to
1. If there are no moments at
d these points are RESTRAINED
g vertically, the vertical com-
R_1 and R_2 must remain constant,
extreme case where M may be
a PIN-CONNECTION and N as
LLERS. Any assumption may be
he magnitudes of the horizontal
at these points as long as the
two equals the sum of the
omponents of R_1 and R_2. It is
o assume these as equal. In
reactions at M and N are TX and YT, respectively. The next step
effect of these reactions at the points O, Q, P and R. The vertical
V_1 and V_2 act as vertical forces at O and P. The horizontal com-
duce bending moments at O and P, and, in effect, horizontal forces
nd R. Taking the left column, the 8 100 lb acting towards the left
he column to the left if not prevented by the joints at O and Q.

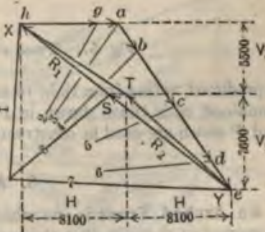


Fig. 55A. Truss with Knee-braces. Force-polygon

If the member MQ is considered to act as a lever with a fulcrum at O, a horizontal force of 8 100 lb acting towards the left at M will produce a pressure, or a force acting from left to right at Q which equals, by the method of moments, the center of moments being at O, 8 100 lb × 7.5 ft ÷ 5 ft = 12 150 lb. At O, in like manner, taking the center of moments at Q, 8 100 lb × 12.5 ft ÷ 5 ft = 20 250 lb

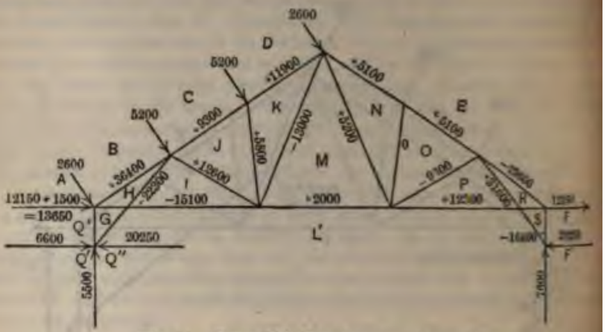


Fig. 56. Truss with Knee-braces. Truss-diagram

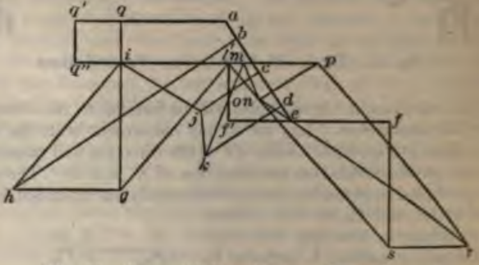


Fig. 57. Truss with Knee-braces. Stress-diagram

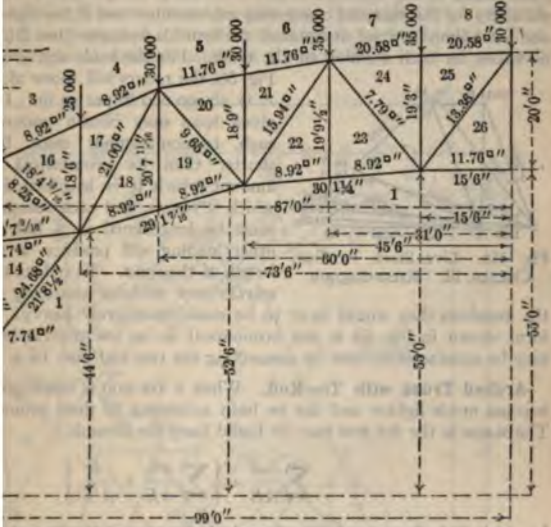
is produced, acting from right to left. These forces are shown in Fig. 56. When combined with those shown in Fig. 55 they give the forces acting at O, Q, and P which are used in constructing the stress-diagram shown in Fig. 57.

8. Arched Trusses

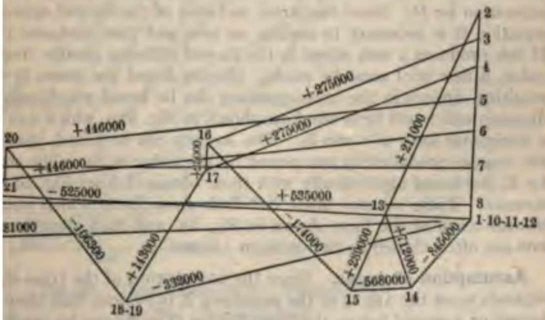
An Arched Truss is one which has the FORM OF AN ARCH and which is so supported at the ends that the reactions produced by vertical forces are vertical. This is usually accomplished by placing PIN-CONNECTIONS at the supports and providing ROLLERS at one end to permit horizontal movement.

Stresses in an Arched Truss. The determination of the stresses in the members of an ARCHED TRUSS is readily accomplished by following the methods given in the previous examples.

Arched Truss with Roller-Support. **Example 35.** In Fig. 58 is shown the left half of an ARCHED TRUSS and the ROLLER-SUPPORT. This truss has the shape and dimensions of a truss in the Live Stock Pavilion, Union Stock mpany, Chicago, Ill. It is discussed in the Engineering News of . The loading shown is symmetrical about the middle of the e each reaction equals one-half the total load. Fig. 58A shows



g. 58. Live Stock Pavilion, Chicago, Ill. Truss-diagram



. Live Stock Pavilion, Chicago, Ill. Stress-diagram for Truss

am for one-half of the truss. The stresses upon the right of the same as those upon the left.

TAL DEFLECTION *of this truss is measured by the movement of* . *This movement is computed in the manner explained for the* *ages 1085–7, by the formula* $D=\Sigma(Sul \div AE)$. Where D is the

HORIZONTAL MOVEMENT, S the stress in any member as given by
shown in Fig. 58A, u the stress in any member produced by th
at the roller end of the truss and acting in a horizontal di
l the length of any member, A the area of any member, E Y
elasticity for the material composing any member and Σ the
and when limits are not designated, the formula indicates that
be taken for each member of the truss. For the loads and
Fig. 58, the rollers will mov
E is 30 000 000 lb per sq in.
given span may obtain u
each tension-member mus
shorter than its geometri
amount which it is length
which it resists, and each co
must be lengthened in a
other loading will produce
length of the span. To redu
DEFLECTION without chang
the members they would have to be made excessively hea
form shown in Fig. 58 is not economical as an ARCHED T
may be satisfactorily used by connecting the two end-pins b

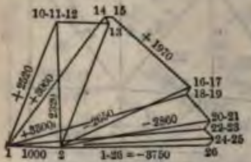


Fig. 58B. Live Stock Pavilion, Chicago, Ill. Stress-diagram

Arched Truss with Tie-Rod. When a TIE-ROD is empl
become much lighter and can be built according to their g
The stress in the TIE-ROD may be found from the formula

$$S_t = \sum \frac{Sul}{AE} \div \left\{ \sum \frac{u^2l}{AE} + \frac{l'}{A'E'} \right\}$$

in which S_t is the stress in the tie-rod, A' the area of the tie-
the tie-rod and E' Young's modulus of elasticity for the
the tie-rod. The other symbols have the significance gi
expression for D. Since the stress and area of the tie-rod a
equation it is necessary to assume an area and then compu
If this produces a unit stress in the tie-rod differing greatly
value, a new trial must be made. Having found the stress
resulting stresses in the truss-members can be found graphi
diagram which will be of the form shown in Fig. 58B, which
a horizontal force of 1 000 lb. The stresses can be found, a
the stresses produced by one pound by the value of S_t. Th
by S_t combined algebraically with those obtained from Fig.
stresses. These stresses differ but little from those which

9. Trussed Arches

russed Arches. The THREE-HINGED ARCH is the simplest RCH, and, as used in buildings, it is usually symmetrical in wo trusses connected by a pin over the middle of the span and each support. The stresses in the truss-members are found aphical methods after the reactions have been determined.

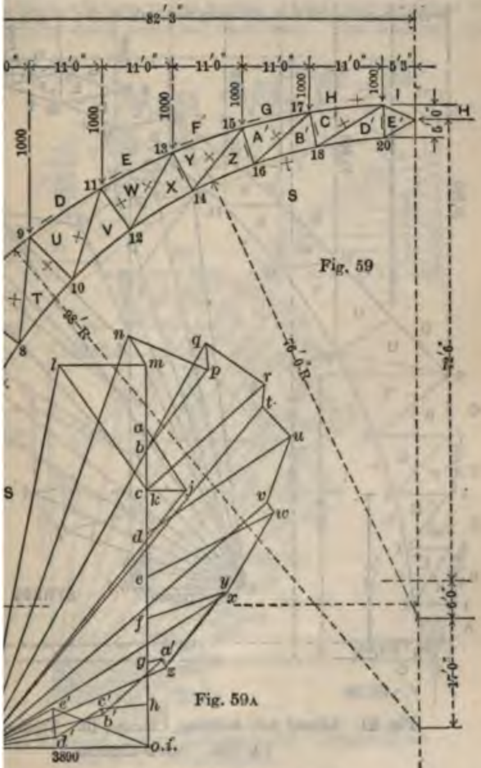


Fig. 59. Three-hinged Arch. Truss-diagram
Fig. 59A. Stress-diagram

RCES are inclined and may be resolved into two components, e other horizontal. For symmetrical loading the two reac- agnitude. The vertical components are each equal to one- ading. *The horizontal components are equal in magnitude racter. The following examples illustrate the methods to ermination of the stresses.*

Trussed Three-hinged Arch. **Example 36.** Fig. 59 shows one-half TRUSSED THREE-HINGED ARCH with a vertical load of 1 000 lb per top- joint. Fig. 59A shows the stress-diagram for this loading; but before it c drawn, the vertical and horizontal reactions at the left support must be d mined. The vertical reaction is (7 × 1 000) + 500 = 7 500 lb or one-half the tical load. The horizontal component or the HORIZONTAL THRUST of the may be found by moments. The center of moments will be taken at the mi

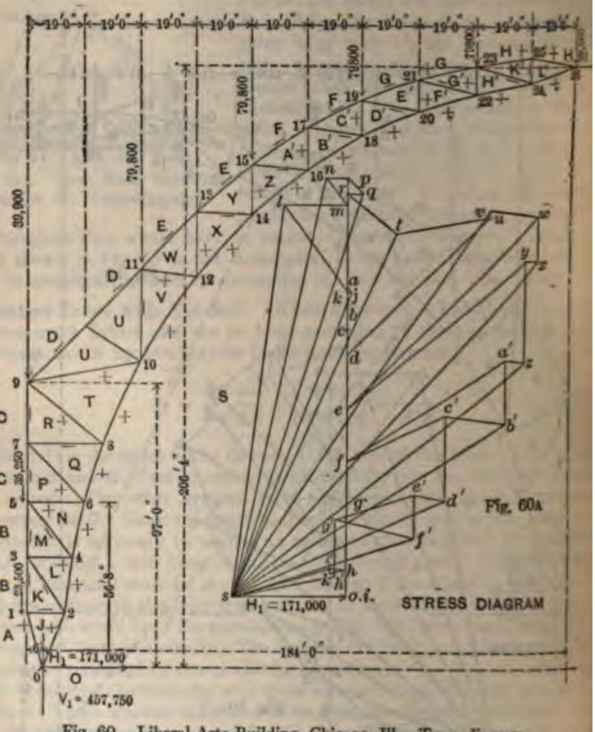


Fig. 60. Liberal Arts Building, Chicago, Ill. Truss-diagram
Fig. 60A. Stress-diagram

pin at the crown as at this point the moment is zero. The equation of mom is $H_1 \times 72.5 + 1\,000\,(5.25 + 16.25 + 27.25 + 38.25 + 49.25 + 60.25 + 71.25)$

$$+ 500 \times 82.25 - 7\,500 \times 78.75 = 0,$$

or

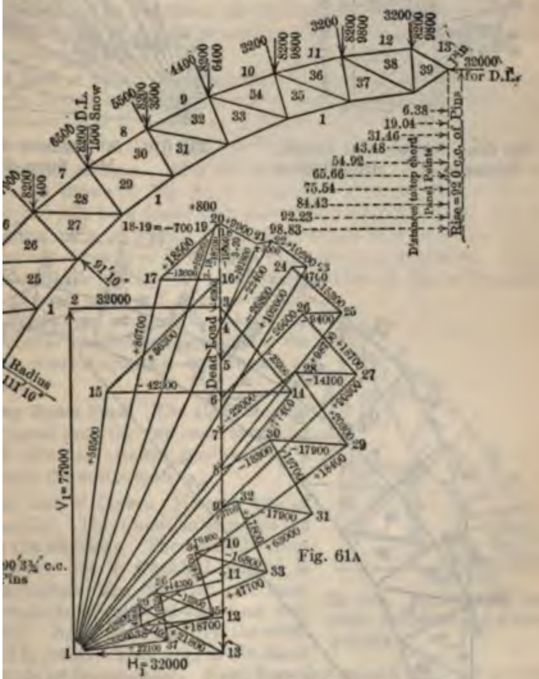
$$H_1 = 281\,750 \div 72.5 = 3\,886 \text{ lb}$$

Having determined V_1 and H_1, the stress-diagram shown in Fig. 59A ca readily constructed. Since the arch is symmetrical, it is necessary to draw one-half the stress-diagram. If the right half of the arch is removed and place a horizontal force applied at the middle pin, the magnitude of this

IZONTAL THRUST H_1, since, for equilibrium, the algebraic sum forces is zero.

ee-hinged Arch. Example 37. Fig. 60 represents one-half of -HINGED ARCH used in the Liberal Arts Building of the Colum- Chicago, Ill., 1893. (See Engineering Record, July 9, 1892.) ress-diagram for the loading shown in Fig. 60.

of Stresses. In Examples 35 and 36 only the effect of vertical onsidered. Where THREE-HINGED ARCHES are employed they to carry dead, snow and wind-loads. The dead and snow-



. 5th Regiment Armory, Baltimore, Md. Truss-diagram
Fig. 61A. Stress-diagram

loads but the snow-load is not symmetrical in all cases. The lly considered as acting normal to the roof. In order to be imum stresses are obtained, the stresses for the following con- must be found and combined.

- D LOAD only,
- *N-LOAD covering left half of roof,*
- *-LOAD covering right half of roof,*
- *LOAD acting normal to roof on left of center,*
- *AD acting normal to roof on right of center.*

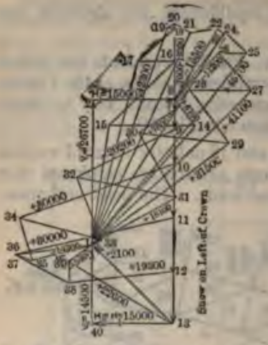


Fig. 61B. 5th Regiment Armory, Baltimore, Md. Stress-diagram

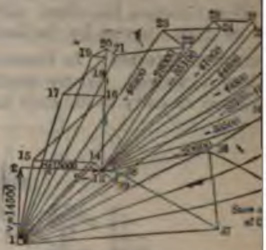


Fig. 61C. 5th Regiment Armory, I
Md. Stress-diagram

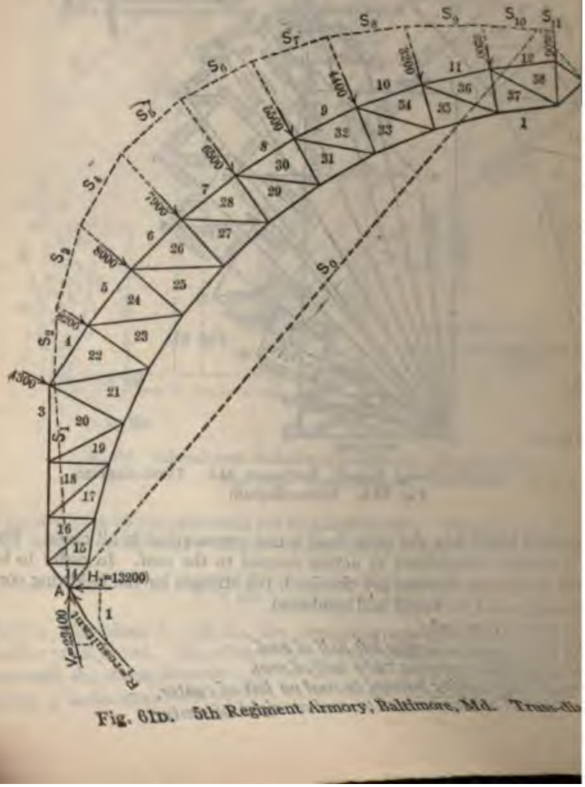


Fig. 61D. 5th Regiment Armory, Baltimore, Md. Truss-di the above conditions of loading are to be found for one-half ombining the stresses those which occur at the same time are ermining maximums. Many engineers do not consider snow ting on the same portion of the roof simultaneously.

e-hinged Arch. Example 38. Fig. 61 shows one-half of a INGED ARCH with the dead, snow and wind-loads indicated at r-chord joints. This form of truss supports the roof of the mory, Baltimore, Md., described in the Engineering Record The stresses for the loadings specified above will be deter- be shown how these are to be combined.

sses. The reactions are obtained by the method used in is 77 900 lb and H_1 32 000 lb. Fig. 61A is the stress-diagram hown in Fig. 61.

Half of Span. Assuming that the snow covers the portion of Fig. 61 and taking the center of moments at the middle pin, nents that V_1 is equal to 26 700 lb and H_1 is equal to 15 000 lb. support the stress-diagram shown in Fig. 61B is readily drawn.

Half of Span. With the snow on the right of the crown, the an shown in Fig. 61 is unloaded. The total snow-load is has just been found that the vertical reaction at the support oading is 26 700 lb; hence the vertical reaction at the other less 26 700 lb or 14 500 lb or V_1 for the case considered. Since e middle pin is zero, V_1 (half the span) less H_1 (rise of the arch) $500 \times 95.16 - H_1 \times 92.0 = 0$; and $.16) \div 92 = 15\,000$ lb which is the ove. As before, beginning at the stress-diagram is constructed as

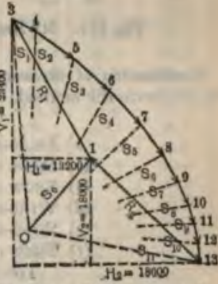


Fig. 61E. 5th Regiment Armory, Baltimore, Md. Force-polygon

Entire Span. The algebraic sum und from the two cases above for give the stresses produced by a g the entire span.

Left of Crown. Here no two of llel. This condition increases the e reactions. These may be com- nts, but a graphical method is enient. The direction and mag- ltant of the wind-forces are first . As shown in Fig. 61E, the wind- n order. Then 3–13 is the direc- de of the resultant. Next, from any point O draw the strings construct the equilibrium polygon shown in Fig. 61D, begin- string S_1 from A, and so on until string S_{11} cuts the line BC he middle pin and the pin at the right support. This is the action at the right support. In Fig. 61E, from 13 draw a line from O a line parallel to S_0 in Fig. 61B, and prolong them until Then 1–3 is the reaction at A and 13–1 that at the right support. into vertical and horizontal components, V_1 equals 23 400 lb, lb, V_2 equals 18 000 lb and H_2 equals 18 600 lb. Fig. 61F shows *n from the left support up to the crown.*

ight of Crown. *Since the reaction at A, Fig. 61D, produced ass through the hinges, or pins A and C,* the stress-diagram will be exactly similar in shape to that shown in Fig. 61c; but the values of and H_2 will be 18 000 lb and 18 600 lb respectively. The stresses will bea direct proportion to the stresses found from Fig. 61c, and hence a new diagr is not necessary.

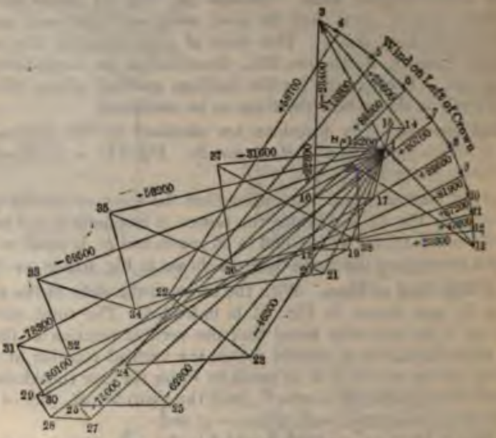


Fig. 61F. 5th Regiment Armory, Baltimore, Md. Stress-diagram

Combination of Stresses. The maximum stresses may now be determin To illustrate the method, consider the lower chord 1–37.

	lb
(*a*) DEAD-LOAD stress,	+ 22 100
(*b*) SNOW on left of crown,	− 14 300
(*c*) SNOW on right of crown,	+ 37 800
(*d*) WIND-LOAD on left of crown,	− 31 600
(*e*) WIND-LOAD on right of crown,	+ 46 900
(*f*) SNOW over all,	+ 23 500
Total stress without wind,	+ 59 900
(*a*) + (*e*),	+ 69 000
(*a*) + (*d*),	− 9 500

The maximum stresses are 69 000 lb compression and 9 500 lb tension, assu ing that the wind and snow-loads are not considered to act on the same side the crown. If no such restriction is made, the maximum stresses are 106 80 compression and 23 800 lb tension. In a like manner the maximum stres each member of the truss is determined. Tables XIX and XX give the M IMUM STRESS for the members shown in Fig. 61.

Stress-Diagrams for Three-hinged Arches. The STRESS-DIAGRAMS in above cases are very difficult to construct owing to the great number of l and the difficulty in drawing them exactly parallel to the lines of the tr diagram. One or more members should be computed as a check on the gr ical work.

Three-hinged Arch with Tie-Rod. The introduction of a TIE-ROD c ing the end-pins of a THREE-HINGED ARCH and placing ROLLERS unde nges the ARCH into a SIMPLE TRUSS composed of three members, fters and a horizontal tie. Under vertical loading, the support- vertical, but for wind-loads the supporting force at the end with- nclined. The stresses in the truss-members are the same as found THREE-HINGED ARCH. The stress in the tie-rod equals the hori- found above at the roller-end for the given loading. The support d is designed for vertical forces only, while the support at the other t the vertical reaction and the total horizontal component of the n the structure, or for roofs the horizontal component of the wind- is very much smaller than the horizontal force which must be the structure is without a tie-rod or a true THREE-HINGED ARCH.

Table XIX. Three-hinged Arch. Chord-Stresses

Thousand pounds

ead ad, 61A	Snow on left of crown, Fig. 61B	Snow on right of crown, Fig. 61C	Snow over all the roof	Wind on left of crown, Fig. 61F	Wind on right of crown,* Fig. 61C	Max. stresses	
						Tension	Compression
25.2	+ 6.1	− 2.1	+ 4.0	+25.6	− 2.6	....	50.8
6.3	− 7.0	−14.0	− 21.0	+32.8	−17.4	30.7	26.5
18.7	−13.2	−21.4	− 34.6	+42.6	−26.5	58.4	23.9
19.0	−13.9	−22.8	− 36.7	+45.8	−28.3	61.2	26.8
22.4	−15.5	−28.0	− 43.5	+58.7	−34.7	72.6	36.3
26.8	−15.8	−35.1	− 50.9	+73.8	−43.5	86.1	47.0
26.6	−11.2	−41.0	− 42.2	+85.3	−50.8	88.6	58.7
22.0	− 3.9	−44.6	− 48.5	+90.7	−55.3	81.2	68.7
13.3	+ 7.8	−44.8	− 37.0	+89.6	−55.6	68.9	76.3
3.7	+20.2	−41.7	− 21.5	+81.9	−51.7	55.4	78.2
6.4	+30.0	−33.5	− 3.5	+67.2	−41.5	35.1	73.6
14.3	+30.0	−20.6	+ 9.4	+47.2	−25.5	11.2	61.5
18.7	+19.3	− 2.8	+ 16.5	+23.3	− 3.5	....	42.0
21.8	+22.6	− 3.3	+ 19.3	+27.8	− 4.1	....	49.6
18.8	− 5.4	+21.9	+ 16.5	− 6.7	+27.2	....	46.0
22.1	−14.3	+37.8	+ 23.5	−31.6	+46.9	9.5	69.0
33.3	−11.3	+51.9	+ 40.6	−53.2	+66.0	19.9	99.3
47.7	+ 2.1	+61.0	+ 63.1	−69.5	+75.6	21.8	125.4
63.0	+18.1	+64.8	+ 82.9	−78.3	+80.4	15.3	161.5
18.4	+31.5	+65.1	+ 96.6	−80.1	+80.7	61.7	130.6
90.3	+41.1	+61.8	+102.9	−75.0	+76.6	....	208.0
98.7	+45.7	+55.8	+101.5	−62.8	+69.2	....	213.6
02.6	+46.0	+48.5	+ 94.5	−46.3	+60.1	....	208.7
01.8	+42.9	+40.0	+ 82.9	−26.0	+49.6	....	194.3
02.3	+42.3	+37.9	+ 80.2	−20.3	+47.0	....	191.6
86.7	+34.8	+29.3	+ 67.7	− 9.6	+36.3	....	157.8
59.5	+22.4	+16.2	+ 38.6	+ 3.9	+20.1	....	102.0
77.4	+29.2	+21.0	+ 50.2	+ 5.2	+26.0	....	132.6

* By proportion, 18 600 : 15 000.

nd no Rollers. *If the* ROLLERS *are omitted and a* TIE-ROD is used, *e tie-rod and the reactions are indeterminate.* They depend upon *idities of the tie-rod and the material composing* the supports. *made very heavy so that its stretch will* be very small when

stressed, the stresses in all members of the structure may be taken the s
found for the condition where rollers are used, and the horizontal compo
the wind-load equally divided between the supports.

Table XX. Three-hinged Arch. Web-Stresses

Thousand pounds

Member, Fig. 61	Dead load, Fig. 61A	Snow on left of crown, Fig. 61B	Snow on right of crown, Fig. 61C	Snow over all the roof	Wind on left of crown, Fig. 61F	Wind on right of crown,* Fig. 61G	Max. str
							Ten-sion
39-38	− 4.9	− 3.8	+ 2.1	− 1.7	−13.7	+ 2.6	16.5
36-37	+10.4	+ 0.3	+14.5	+14.8	−18.5	+18.0	8.1
34-35	+14.5	+ 8.3	+13.0	+21.3	−18.6	+16.1	4.1
32-33	+17.8	+12.7	+10.6	+23.3	−15.4	+13.1	....
30-31	+19.7	+13.8	+ 7.6	+21.4	−10.7	+ 9.4	....
28-29	+20.3	+12.3	+ 4.7	+17.0	− 4.9	+ 5.8	....
26-27	+18.7	+ 9.3	+ 1.2	+10.5	+ 3.4	+ 1.5	....
24-25	+15.3	+ 5.4	− 1.8	+ 3.6	+10.8	− 2.2	....
22-23	+10.2	+ 1.5	− 5.1	− 3.6	+19.0	− 6.3	....
20-21	+ 9.9	+ 3.5	+ 2.0	+ 5.5	+ 2.4	+ 2.5	....
18-19	− 0.7	− 9.3	− 2.7	−12.0	+ 6.6	− 3.3	13.3
16-17	−13.6	− 6.8	− 8.1	−14.9	+10.9	−10.0	30.4
14-15	−42.3	−15.9	−11.4	−27.3	+ 2.8	−14.1	72.3
37-38	− 7.1	+11.2	−22.0	−10.8	+29.8	−27.3	23.2
35-36	−12.9	− 3.5	−16.3	−19.8	+24.9	−20.2	36.6
33-34	−16.8	−15.8	−10.4	−26.2	+18.8	−12.9	45.5
31-32	−17.9	−19.2	− 4.2	−23.4	+10.0	− 5.2	42.3
29-30	−17.9	−17.0	+ 0.6	−16.4	+ 1.4	+ 0.7	34.9
27-28	−14.1	−11.6	+ 5.0	− 6.6	− 7.4	+ 6.2	25.7
25-26	− 9.4	− 5.3	+ 8.7	+ 3.4	−17.0	+10.8	26.4
23-24	− 4.7	+ 0.4	+11.0	+11.4	−23.3	+13.6	28.0
21-22	+ 3.0	+ 5.2	+13.1	+18.3	−29.5	+16.2	26.5
19-20	+ 0.8	+ 1.6	+ 3.1	+ 4.7	− 7.4	+ 3.8	6.6
17-18	+18.5	+ 9.2	+10.9	+20.1	−14.8	+13.5	....
15-16	+36.3	+16.8	+17.8	+34.6	−19.1	+22.1	....

* By proportion, 18 600 : 15 000.

Changes in Temperature do not seriously affect the stresses in the
of a TRUE THREE-HINGED ARCH, or one with a tie-rod and rollers at
as the change in geometrical shape is quite small. For the arch with
and no rollers, the effect of changes in temperature may affect the su
forces if the tie-rod is not so protected that it will change but little
average temperature. In most structures this is the case as the tie-ro
under the floor of the building.

The Two-hinged Arch differs essentially in construction from th
HINGED ARCH in having only two pins or hinges which are placed at the
Fig. 62 shows the form of truss which will be used in explaining the m
finding the stresses in the members of the truss.

Supporting Forces. The SUPPORTING FORCES are inclined but c
solved into vertical and horizontal components. The vertical compo
readily found as they are the same as for a simple truss on two supp
horizontal components depend upon the AREAS of the members and
OF ELASTICITY when the dimensions of the truss and the loading

ontal Thrust for Vertical Loads. This can be found from the formula

$$H_1 = \sum \frac{Sul}{AE} \div \sum \frac{u^2 l}{AE}$$

e symbols have the significance given on page 1086. But this contains own area *A* for each piece. For a preliminary trial the procedure is as In the truss shown in Fig. 62, divide the span into twenty equal parts he centers of the divisions erect verticals. Through the points on these midway between the chords of the truss, draw a smooth curve as shown. will be designated the AXIS OF THE ARCH. Number the points desig-

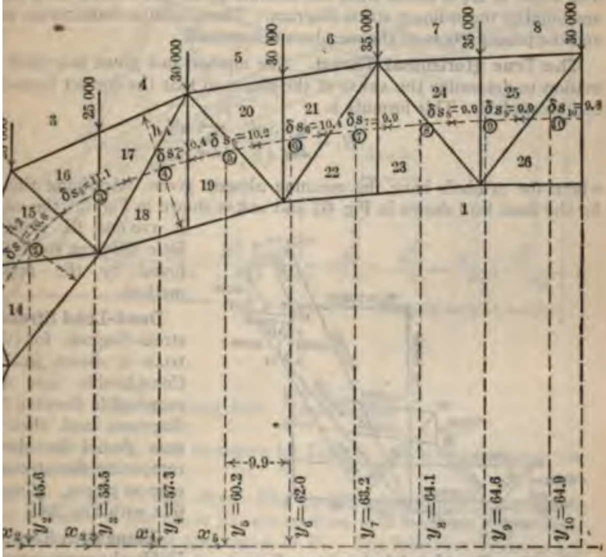


Fig. 62. Two-hinged Arch. Truss-diagram

ove, 1, 2, 3, etc., as shown in Fig. 62, and let *x* and *y* be their coördi- h the left support as the origin. Scale the length of the curve between rs of the divisions so that *y* is practically the ordinate of the center of length of curve, and call this length of the curve δs. On a radial line oint numbered 1, 2, 3, etc., scale the distances between the upper and ords, calling the distance *h* and compute $\frac{1}{2} h^2 = I$, which expresses, ately, the MOMENT OF INERTIA of the section when the chord-areas and the web-members are neglected. Let *M* represent the BENDING at any point having the abscissa *x*, of the loads, considering the truss as beam on two supports; or, for a single load *P*, $M = Rx - P(x - a)$, reater than *a*, where *a* is the distance of the load *P* from the left sup- hen if $\delta s \div EI$ *is represented by* ϕ *the* HORIZONTAL THRUST can be found *formula,*

$$H_1 = \Sigma M y \phi \div \Sigma y^2 \phi$$

For the vertical loading shown in Fig. 62, the value of H_1 is 108 000 being one-half the total load, is 195 000 lb. The stresses in the mem truss can now be found by the usual graphical method. The snow-l must be treated in a like manner. The computations are considera since $\Sigma y^2\phi$ remains unchanged, regardless of the loading.

Wind-Loads. For WIND-LOADS the process is not changed very value of M is the moment of the wind-loads, assuming the truss a the right support and on ROLLERS at the left support. The value of vertical, is found by taking the sum of the moments of the wind the hinge at the right support and dividing this by the length The value of H_1 is found from the formula given above, and then are found by the ordinary stress-diagram. The MAXIMUM STRESSES a and the proper AREAS of the members determined.

The True Horizontal Thrust. The method just given is a c mation to determine the AREAS of the pieces so that the correct fo can be applied. This formula is

$$H_1 = \sum \frac{Sul}{AE} \div \sum \frac{u^2l}{AE}$$

where the symbols have the meaning already given. Applying for the dead load shown in Fig. 62 and AREAS shown in Fig. 58, th is 110 600 lb, w little different fro found by the method.

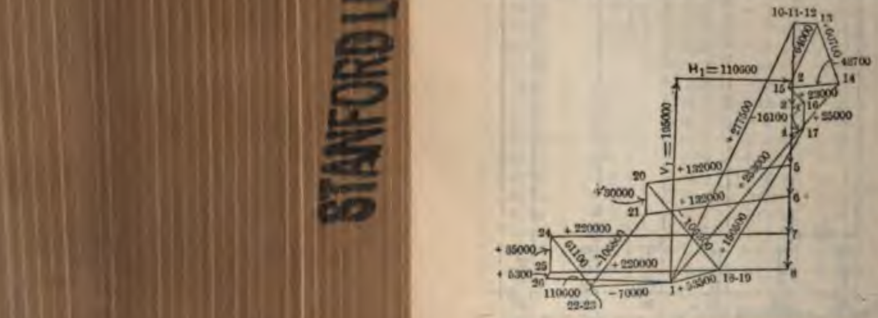


Fig. 62A. Two-hinged Arch. Stress-diagram

Dead-Load St stress-diagram fo LOAD is shown i Considerable ca exercised in drawi diagrams, and t ness should be computing the st or more pieces. C 62A with Fig. 58

Changes in T Unlike the THREE- the TWO-HINGE affected by CHANGES IN TEMPERATURE and the stresses which are such changes must be provided for. $V_1 = O$ and H_1 is determin formula

$$H_1 = et^\circ L \div \sum \frac{u^2l}{}$$

has no hinges and is a type which is seldom employed by uss-form. The rigid analysis of a TRUSSED FIXED ARCH is ıs, so a few formulas will be given, necessary for the solution LID WEBS, such as PLATE-GIRDER ARCHES. These formulas truss-forms, where the chords are approximately parallel, or. Midway between the top and bottom chords draw a l the ARCH-AXIS, and designate the distance between its ends THE AXIS. Divide the span into n equal parts and at the visions draw perpendiculars until they cut the arch-axis.

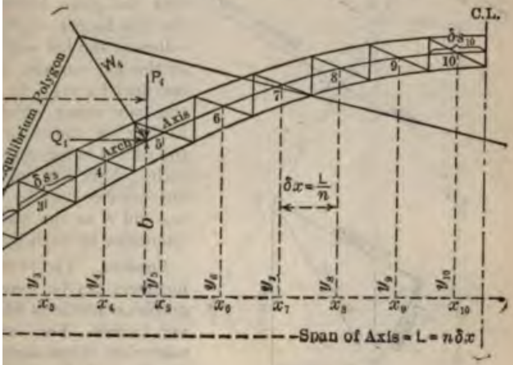


Fig. 63. Fixed Arch. Truss-diagram

1, 2, 3, etc., as shown by Fig. 63, which also indicates the oyed.

H_1, V_1 and H_1y_1. The equilibrium-polygon for a single wn in Fig. 63, in its true position with reference to the arch- the point of application of H_1. The following formulas are ıations for arches having a rise greater than one-eighth the

$$_1 = \Sigma m_x A'' \div \Sigma y A''$$

$$'' = K\left\{y - \frac{\Sigma yK}{\Sigma K}\right\} \qquad K = \frac{\delta s}{EI}$$

$$\Big\} = H_1 \frac{\Sigma yK}{\Sigma K} - \left\{\frac{\Sigma m_x K}{\Sigma K} \pm \frac{\Sigma m_x K\,(z-n)}{n^2 \Sigma K - \Sigma z^2 K}\, n\right\}$$

$$_1 = \frac{H_1y_1 - H_2y_2}{L} + r_1 \qquad y_1 = \frac{H_1y_1}{H_1}$$

ı from A when H_1y_1 is negative. Σ is the sum of quantities ı point on the arch-axis numbered *1, 2, 3, . . . n.* For

$$K = \left(\frac{\delta s}{EI}\right)_1 + \left(\frac{\delta s}{EI}\right)_2 + \left(\frac{\delta s}{EI}\right)_3 + \text{etc.}$$

I is the moment of inertia of the chords about an axis midway betwe The sections of the chords are to be taken on radial lines passing through 2, 3, etc. x and y are the coördi the points 1, 2, 3, etc, in Fig. 63

$$x = z\frac{\delta z}{2} = z\frac{L}{2n}$$

m_x is the moment at the poin arch-axis having the abscissa x, that the given loading is supp the axis hinged at the right en rollers at the left end. r_1 is the at the left support under the c specified for m_x. In the above the only terms which depend loading are those containing the others being constant for arch. While but one load has any number may be used by c m_x and r_1 as the sum of the quantities for each load.

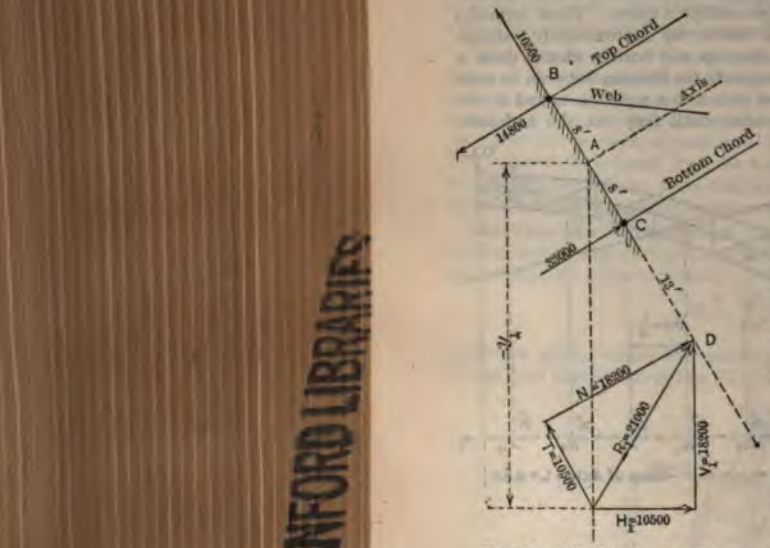


Fig. 64. Fixed Arch. Reactions

Stresses. The STRESSES in members can be found by the graphical methods when H_1, V_1 are known. For example, as numerical values shown in Fig. resultant of V_1 and H_1 is res two components parallel and ular to the bottom-chord m the support. Then T must upper-chord joint as shown. The two reactions parallel to the bott are found by moments. The stress-diagram can now be drawn begin these forces and proceeding until the right support is reached.

Symmetrical Loading. When the loading is symmetrical, H_1y_1 = hence $V_1 = r_1$. Also

$$H_1y_1 = H_1\frac{\Sigma yK}{\Sigma K} - \frac{\Sigma m_xK}{\Sigma K}$$

Changes of Temperature. For temperature-changes,

$$H_t = et^\circ L \div \Sigma yA''$$

$$H_1y_1 = H_2y_2 = H_t\frac{\Sigma yK}{\Sigma K}$$

$$y_1 = \frac{\Sigma yK}{\Sigma K} \qquad V_1 = 0$$

10. Arches with Solid Ribs

Arches with Solid Ribs. While this chapter considers TRUSSE may not be out of place to briefly consider ARCHES HAVING SOLID computations for V_1, H_1 and H_1y_1 remain unchanged, excepting the moment of inertia of the radial section of the rib at points 1, 2

ses. If x and y are the coördinates of any point on the gravity-
b, which should coincide with the arch-axis, the bending moment
is, for each load,

$$M_x = H_1y_1 + V_1x - H_1y - P\overset{x>a}{(x-a)} - Q\overset{x>a}{(y-b)}$$

ve when y_1 is measured below A in Fig. 64.

$$S = \frac{M_x c}{I}$$

e distance from the gravity-axis to the outermost fiber. For the
EE-HINGED ARCHES, $H_1y_1 = 0$.

ar. Let H_x be the algebraic sum of all the horizontal components
the section, V_x the algebraic sum of all the vertical components
the section and θ the angle which the radial section, upon which
vanted, makes with the vertical. Then $T_x = V_x \cos\theta - H_x \sin\theta$.

d Parabolic Arch. If the center line of the SOLID RIB is a PAR-
$EI \cos\theta$ is a constant, the following simple formulas give the
nd H_1:

$$-k) - Q\frac{4f}{L}k(1-k)$$

$$P[k(1-2k^2+k^3)] - Q\left\{1 - \frac{k}{2}[5(1-k-2k^2+4k^3) - 8k^4]\right\}$$

$$EI_0 et^\circ$$

$\div L$ (Fig. 63), f is the rise of the axis, P is the vertical load acting
e horizontal load acting from left to right and I_0 is the moment of
section of the rib at the crown.

abolic Arch. In like manner the following formulas apply for
out hinges:

$$P(1-k)^2(1+2k) - \frac{12f}{L}Q(k-k^2)^2$$

$$\frac{15L}{4f}Pk^2(1-k)^2 - Q\{1 + k^2(-15 + 50k - 60k^2 + 24k^3)\}$$

$$\frac{L}{2}Pk(1-k)^2(5k-2) - fQ\{2k(1-k)^2(2-7k+8k^2)\}$$

$$\frac{45}{4f}EI_0 et^\circ \qquad H_1y_1 = \frac{15}{2f}EI_0 et^\circ$$

the factors containing k in the above formulas are given in tabular
eatise on Arches.*

rches, with solid ribs of constant cross-section and the center line
ircle, may be considered by using formulas somewhat similar to
or PARABOLIC ARCHES but very much longer and more complex.
tables for their *solution are given in the treatise on arches referred*

Arches, by Malverd A. Howe, John Wiley & Sons, Inc., New York.

11. Influence-Lines for Simple Beams and Trusses

An Influence-Line is a line showing the variation in any function section of a beam or for any member of a truss, caused by a single load across the span. For convenience the LOAD is usually considered as UNIT

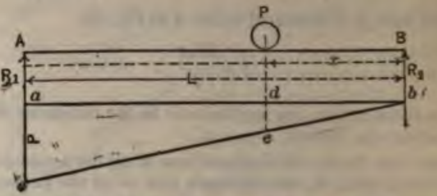


Fig. 65. Influence-lines. Reactions for Beams

Reaction for a Single Load. If the load P, Fig. 65, moves from A B, the left reaction, when P is distant x from B, is expressed algebraic $R_1 = Px \div L$, which is an equation of a straight line. If $x = 0$, $R_1 =$ if $x = L$, $R_1 = P$. If we make $ac = P$ and draw the two straight lines ab the ordinate ***de*** immediately below P is the value of R_1 for this positio If $ac =$ unity, then $R_1 = P\ (de)$.

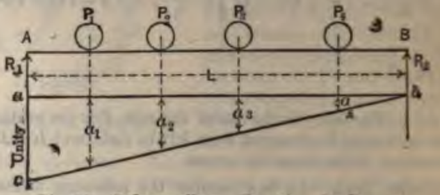


Fig. 66. Influence-lines. Reactions for Beams

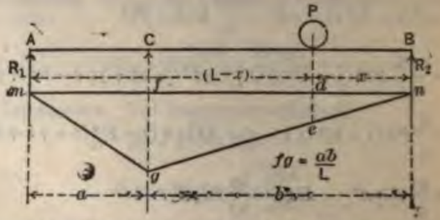


Fig. 67. Influence-lines. Moments for Beams

Reaction for More than One Load. The reaction for any nu concentrated loads can be found as shown in Fig. 66.

$$R_1 = P_1a_1 + P_2a_2 + P_3a_3 + P_4a_4$$

Bending Moment for a Single Load. The moment at C, Fig. 67, is on the right of C, is $M = R_1a = \frac{P x a}{L}$. When the load is at C, M t B, $M = 0$. For all positions of P upon the left of C, $M = R_2b$ $\frac{b}{}$. When P is at A, $M = 0$, and when at C, $M = P\frac{(L-b)b}{L}$

If in Fig. 67 the figure *mng* is drawn with $fg = \frac{(a)(b)}{L}$, then the for any load in any position is P (*de*).

Moment for Any Number of Concentrated Loads. The the point C for the loading shown in Fig. 68 is $M = P_1a_1 + P_2a_2$

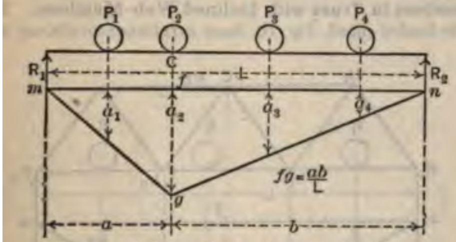


Fig. 68. Influence-lines. Moments for Beams

$_4$. This gives the moment at C for a given position of the loads, ot necessarily the GREATEST MOMENT which these loads may cause, r position may cause a greater moment. The greatest moment at when some concentration is at C. Let P be this concentration and

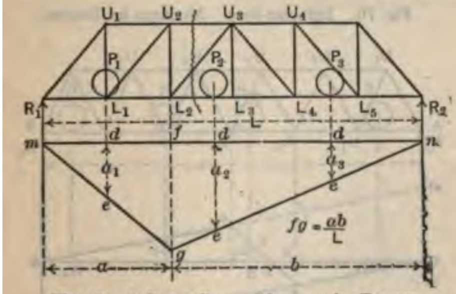


Fig. 69. Influence-lines. Moments for Trusses

be divided into two parts, nP and mP so that $n + m = 1$, and n is zero and less than 1. The maximum moment at C will occur when

$$\frac{P_1+P_2+P_3+P_4}{L} = \frac{P_1+nP_2}{a}$$

in the beam where any given moving load causes the GREATEST MENT is so situated that the *middle of the span is half-way between nter of gravity of the load. Since a concentration will always be a few trials will determine the proper concentration to use.* For *qual concentrated loads should be placed on the beam so that*

the middle of the span is at the quarter-point between the concentrati
MAXIMUM MOMENT falls under the concentration nearer the middle of the

Chord-Member in Truss with One Set of Web-Members Vertical. Fig. 69 the top chord member U_2U_3 has its center of moments at L_2 bottom chord member L_1L_2 at U_2. The INFLUENCE-DIAGRAM for the at L_2 and U_2 is precisely the same as shown in Fig. 67. The moment pro by any load P is P (de). As long as one set of web-members is vertic INFLUENCE-DIAGRAM will be identical with that shown in Fig. 69, reg of the inclination of the diagonals or the chord-members.

Chord-Members in Truss with Inclined Web-Members. The m at points in the loaded chord, Fig. 70, have INFLUENCE-DIAGRAMS identic

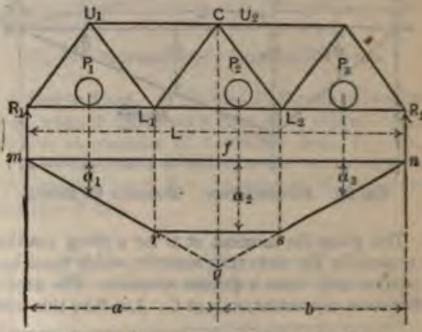


Fig. 70. Influence-lines. Moments for Trusses

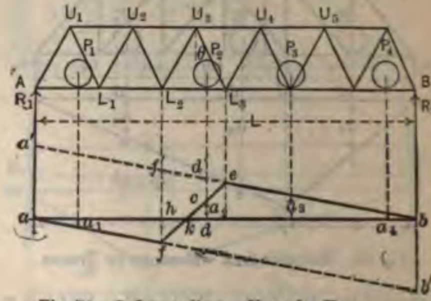


Fig. 71. Influence-lines. Shear for Trusses

that shown by Fig. 69. For the unloaded chord a slight modification made. For example let U_2 be a center of moments, then if the loads beam, mgn would be the INFLUENCE-DIAGRAM (Fig. 70). For all loa left of L_1 and on the right of L_2 the diagram is correct and the mome $= P_1a_1$ and P_3a_3. For loads between L_1 and L_2 draw the line rs. The at U_2 is P_2a_2.

Web-Members of Trusses with Parallel Chords. Fig. 71. in U_3L_3 equals the shear in the panel L_2L_3 multiplied by the seca

IAGRAM will be drawn for the shear. For any load between L_3 iear in this panel equals R_2; hence, with *ab* as a reference-line, *ba'* NCE-LINE for R_2 and the shear is P_4a_4, P_3a_3, etc., until the point L_3 In like manner *af* is the INFLUENCE-LINE for R_1 and the shear for left of L_2 is P_1a_1. The shear for the loads P_2 between L_2 and L_3 amount of P_2 which is transferred to L_2. The INFLUENCE-DIAGRAM ons of P_2 on a span L_2L_3 is *ff'e*. The shear in this panel due to P_2 s P_2 (*d'c*) or P_2a_2. A load at *k* produces no shear in the panel.

CHAPTER XXVIII

DESIGN AND CONSTRUCTION OF ROOF-TRUS

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1. Design of Wooden Trusses

Proportioning the Members. In Chapter XXVII it has been sh the STRESSES in the members of a truss, supporting known LOADS, may The next step is to PROPORTION THE MEMBERS for the stresses which t to resist. The methods employed and the ALLOWABLE UNIT STRESSES in detail in Chapters XI to XVI, inclusive. For example, tension-me considered on pages 385 to 400; steel strut-beams and tie-beams on and 572; and wooden strut-beams and tie-beams on page 633. As of convenience the UNIT STRESSES used in this chapter are given in the table in a condensed form. White pine is here used for the wooden t

Table I. Allowable Unit Stresses Used in Truss-Design*

Material	Kind of stress	Safe u lb pe
White pine.........	Tension with the grain......................	
"	Tension across the grain.....................	
"	Compression on end-fibers..................	1
"	Compression across the grain................	
"	Compression across the grain, round pins..	
"	Columns† under 15 diam long...............	1
"	Shear with the grain........................	
"	Shear across the grain......................	
"	Transverse, fiber-stress....................	
Wrought iron.......	Bolts in tension...........................	12
"	Bolts in shear.............................	7
"	Bolts in bearing...........................	15
"	Bolts in bending, fiber-stress..............	15
Rolled steel........	Bolts in tension...........................	16
"	Bolts in shear.............................	10
"	Bolts in bearing...........................	20
"	Bolts in bending, fiber-stress..............	20
"	Beams in bending, fiber-stress..............	16
"	Beams in shear.............................	9

* These must be modified, when necessary to comply with building laws. is used for the examples in this chapter because of the difficulties in makin owing to the relative softness of the wood. If one can design a truss in white have no trouble with the design of trusses constructed with other kinds of woo

† See, also, Table I, page 449, and Table XVI, page 647.

Inclined Surfaces of Wood. The normal intensity of the str clined surfaces may be found from the empirical formula

$$r = q + (p - q)(\theta/90)^2$$

ıls the permissible normal unit stress on this inclined surface, q
ıe fibers, p that on the end of the fibers and θ the angle the inclined
s with the direction of the grain. For white pine this gives

$$r = 200 + \theta^2/9$$

ns on End-Fibers.* For all practical purposes the permissible
ay be taken as the mean of p and q; or, for white pine

$$\tfrac{1}{2}(p + q) = 650 \text{ lb per sq in}$$

Columns over Fifteen Diameters Long. The formula † used in
and considered amply conservative by many engineers is the
oved by the American Railway Engineering and Maintenance of
tion in 1907. For white pine this formula is

$$S_1 = S\,(1 - l/60\,d) = 1100\,(1 - l/60\,d)$$

ıe permissible unit stress, S = the permissible compression on the
= the length of the column in inches and d the least dimension of
ion of the column in inches.

ımns. For the shapes used in roof-trusses, the formula advocated
vler in his specifications for roof-trusses is used in this chapter:

$$S_1 = 12\,500 - 500\,l/r$$

ıe permissible unit stress, l = the length of the column in feet, and
radius of gyration of the cross-section of the column.

. The truss shown in Fig. 1, which is the queen truss shown in
3 and 54 in Chapter XXVII, is considered for this example. The
ı in the following table are used. The members RR are wrought-
ods, not upset at the ends; and all other members are of white
of the members in this truss is subject to transverse stress, so
ı and compression only, have to be considered:

. Stresses and Dimensions for the Truss Shown in Figure 1

er	Stress in pounds	Dimensions
....	+21 150	6 by 6-in white pine
....	+18 900	6 by 6-in white pine
....	+13 200	6 by 6-in white pine
....	+ 6 450	4 by 6-in white pine
....	+ 5 100	4 by 6-in white pine
....	−17 150	6 by 8-in white pine or Three 2 by 8-in pieces with ¾-in bolts, 2 ft on centers
....	−12 750	
....	− 8 410	One 1¼-in round rod

ds, Fig. 1. The tension in each rod is 8 410 lb. If the permis-
12 000 lb, the section-area of each rod is 8 410 ÷ 12 000 = 0.70 sq in.
of a 1⅛-in rod is 0.694 sq in; and of a 1¼-in rod, 0.893 sq in. The
uld answer but the 1¼-in rod is preferred.

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same unit stresses are used for flat and curved surfaces, Tables VII and
to 431, of Chapter XII may be used.

mulas and Tables based upon them, see Chapter XIV, pages 449 to 452.

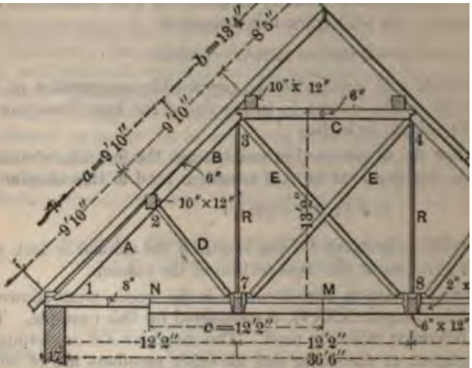


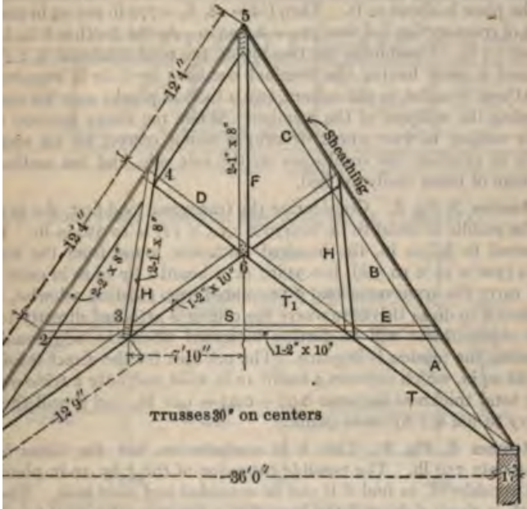
Fig. 1. Queen Truss. (See, also, Figs. 4A, 10, 13 and 16 and
3, 12, 53 and 54)

Member *C*, Fig. 1. The stress in this member is 13 200 l
length, 12 ft. In this case $l/d = 24$, when $d = 6$ in; $S_1 = 6$
required section-area is 13 200/660 = 20 sq in, and hence
used. The top-chord should have one dimension constant
the making of good connections at the joints.

Braces, Fig. 1. The stress in the brace *D* is 6 450 lb
length about 9 ft. A 4 by 6-in timber is first tried. Here
lb per sq in. The required area, therefore, is 10.7 sq in a
answers the purpose; but for additional stiffness and conve

lted together by a pair of bolts every 2 ft of their length. If 24-ngths are used, the joints of the strands will be about 10 ft apart.

For this example the truss illustrated in Fig. 2, which is the shown in Figs. 4 and 24, Chapter XXVII, is considered. The for dead load, wind and snow were found in Chapter XXVII and the following table. The rafters and the bottom chord support



. Scissors Truss. (See, also, Chapter XXVII, Figs. 4 and 24)

the joints and consequently must resist CROSS-BENDING stresses ECT STRESSES. The load on each piece is given in the table under NSVERSE.

. Stresses and Dimensions for the Truss Shown in Figure 2

er	Stress, lb	Transverse load, lb	Dimensions, white pine
....	+8 000	1 000	Two 2 by 8-in planks
....	+6 600	1 320	Two 2 by 8-in planks
....	+1 890	....	One 2 by 10-in plank
....	+ 750	....	One 2 by 10-in plank
....	−4 350	....	Two 1 by 8-in planks
....	−2 530	....	Two 1 by 8-in planks
....	−1 875	470	One 2 by 10-in plank
....	−5 400	384	One 2 by 10-in plank
....	−1 875	....	One 2 by 10-in plank

Fig. 2. *The piece B rather than the piece A is considered,* as it is *onger. The total vertical load on the piece acting* as a beam is *horizontal span is about 8 ft.* The bending moment at the center

creasing the stiffness of the member. While the above
piece subject to TWO KINDS OF STRESS is not correct fo
occur in practice, the results are on the safe side, and th
vantage of being easily applied.

Member S, Fig. 2. Considering the transverse load firs
at the middle is found to be $\frac{1}{8}(470 \times 15.5 \times 12) = 10\ 930$
assumed to be 10 in, the required thickness, found fro
$= \frac{1}{6}(700 \times 10 \times 10 \times b)$, is 0.94 in; or, a board 1 by 10 i
will carry the transverse load if prevented from twisting
tendency to do in this case where the ceiling is attached
The side-stiffness will be further increased when the
resisting the tension is in place. The net area for the di
is 2.68 sq in, which requires a board 10 in wide and only
The total thickness becomes $0.94 + 0.27 = 1.21$ in, and i
essary to use a 2 by 10-in plank.

Member E, Fig. 2. This is in compression, but th
being only 750 lb. The possible extension of the 2 by 1
next considered, to find if it can be extended and used h
length is about 6 ft, and the least dimension 2 in; henc
per sq in and the required area of the cross-section bec
The 2 by 10-in piece is therefore ample.

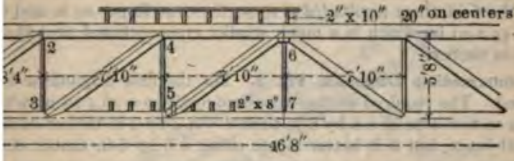
Members T and T_1, Fig. 2. Inspection shows that
quite sufficient for these pieces.

Member D, Fig. 2. The unsupported length is about
in, $l/d = 42$, $S_1 = 330$ lb per sq in and the required sect
5.73 sq in. A piece 2 by 10 in is more than sufficient; b
a simple prolongation of the pieces T and T_1, a piece of

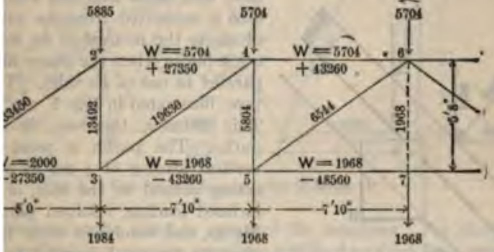
Members F and H, Fig. 2. For the piece F a net ar
sq in is required; or a board 1 by 8 in in cross-section m

resses are shown in Fig. 4. The figures preceded by the letter
ransverse loading of the pieces.

Fig. 3. These are assumed to be wrought iron and not upset.
492 lb (Fig. 4), using the unit stress 12 000 lb per sq in, the net
iired is 1.16 sq in, which is provided by a 1½-in rod (Table II,
the stress 5 804, the net section-area required is 0.48 sq in, which
1-in rod. . The stress in the middle rod is so small that the rod



Howe Truss. (See, also, Figs. 4, 9A, 9B, 14, 15 and 17A)



Howe-truss Diagram. Stresses in Truss Shown in Fig. 3

ttle over ½-in in diameter. This would appear light so a ¾-in

g. 3. As this is to be uniform in size from end to end, a
s considered in determining its size. It is assumed that the
or the transverse load the center moment is ⅛ (5 704 × 7⅚ × 12)
From the equation, 67 022 = ⅙ (700 × 10 × 10 × b), the breadth
r the compression, 43 260 lb, the least dimension may be
as the rafters prevent SIDE BUCKLING for the unsupported
Then $l/d = 9.4$, $S_1 = 1\ 100$ lb per sq in (note that the piece is
liameters long) and the required section-area becomes 39.3 sq in.
in, the thickness is 3.93 in. The total thickness of the piece now
.93 = 9.67 in; and hence a 10 by 10-in piece may be used. Since
ection size of a nominal 10 by 10-in piece is about 9½ by 9½ in,
ber is used and the 12-in dimension is placed vertical.

Fig. 3. Considering the piece at the middle, with a tensile
lb, it is found that the net area of the cross-section must be
f the piece is 12 in deep, the thickness is 5.78 in. The bending
niddle of the piece produced by the load 1 968 lb, is ⅛ (1 968 ×
4 in-lb. *This moment requires a piece 12 in deep and* 1.28 in
thickness now becomes 5.78 + 1.28 = 7.06 in. To allow for holes
ecessary to increase this to at least 10 in, making the bottom

chord 10 by 12 in. A single piece of this size, nearly 50 ft lon
obtain; so at least one splice is necessary. If planks are substit
six 2 by 12-in pieces to give an equivalent area.

Inclined End-Post, Fig. 3. The stress in this post is 33 450
supported length about 9.75 ft. An 8 by 10-in piece is tried
dimension being the same as one dimension of the chords. Th
$S_1 = 825$ lb per sq in and the required area of cross-section become
40.54 sq in, which is about one-half the cross-sectional area of an 8
If $d = 6$ in, there results $l/d = 19.50$, $S_1 = 740$ lb per sq in and th
= 45.2 sq in, which is a much smaller cross-sectional area than
10-in timber.

Intermediate Diagonals, Fig. 3. For the first diagonal a 6 b
tried. The required section-area is 19 630/740 = 26.5 sq in, whic
the section-area of a 6 by 6-in timber. A 4 by 4-in timber could
next brace, but it is better to use either a 6 by 6-in timber or o

Purlins, Figs. 1 and 4A. While the stresses given for the m
truss shown in Fig. 1 are based upon a vertical loading coverin
dead load, snow-load and wind-l
load is separated from the othe
illustrate the method to be follo
ing a purlin when the plane of t
parallel to one of its sides. The
type illustrated in Fig. 1, are 15
This distance, therefore, is th
purlin. The purlin at joint 2,
vertical loading of 4 500 lb an
acting normal to the roof, of 2
inclined loading, resolved parall
Fig 4A, and combined with the
gives for the total, parallel to d,
= 5 900 lb; and for that para
lb. If then, loads are assumed
the center of gravity of the purli
produce both tension in the fiber at B and compression diagonal
The bending moment at the middle of the purlin due to the ver
(5 900 × 15 × 12) = 132 750 in-lb; and that due to the horizo
(1 400 × 15 × 12) = 31 500 in-lb. It is assumed that $b = 8$ in
Then the fiber-stress at B, due to the first moment is

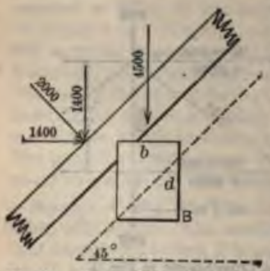


Fig. 4A. Purlin-design for Joint 2 of Truss Shown in Fig. 1

$$S' = 6 \times 132\,750/bd^2 = 996 \text{ lb per sq in}$$

The fiber-stress at B, produced by the second moment is

$$S'' = 6 \times 31\,500/b^2d = 295 \text{ lb per sq in}$$

The total fiber-stress is 996 + 295 = 1 291 lb per sq in. This is
of the permissible fiber-stress in the most conservative practice an
building laws for long-leaf yellow pine or white oak. If a 10 b
is used the fiber-stress is 986 lb per sq in, and if the piece is 10
comes 710 lb per sq in.

2. Design of Steel Trusses

General Considerations. The members of the ordinary
TRUSSES are composed of two rolled ANGLES placed back to b
joints each piece is connected to GUSSET-PLATES by RIVETS.

gle permissible in good practice is 2½ by 2 by ¼ in; and while ⅝-in ften used, it is better to use ¾-in rivets, which are the largest that l in a 2½-in leg of an angle. As in wooden trusses, it is economical same sizes for all members which are in the same straight line, but always done.

4. Fig. 5 shows a FAN TRUSS of the form and dimensions of a truss pporting the roof of a machine-shop. The loading is light and con-

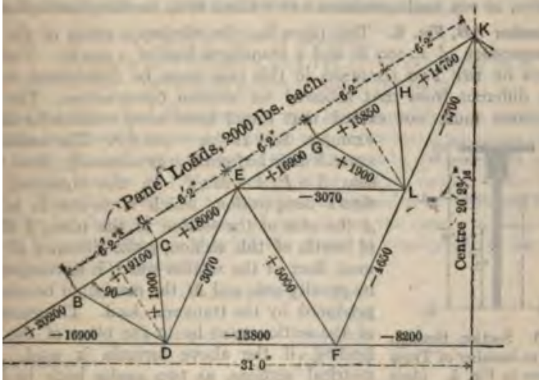


Fig. 5. Fan-truss Diagram with Stresses

he stresses are quite small in many of the members. For conven- resses, lengths of compression-members and final sections are arranged orm.

Stresses and Dimensions for the Half-Truss Shown in Figure 5

r	Stress, lb	Approximate length, in	Net area required, sq in	Make-up of member
.....	−16 900	....	1.06	Two 2½ by 2 by ¼-in angles Net area = 1.70 sq in
.....	−13 800	....	....	
.....	− 9 200	....	....	
.....	− 4 650	....	....	
.....	− 7 700	....	....	
L...	− 3 070	....	....	
.....	+20 200	72	....	Two 2½ by 2½ by ¼-in angles, and one 10 by ¼-in plate
.....	+ 5 000	144	....	Two 2½ by 2 by ¼-in angles
.....	+ 1 900	72	....	

AD, **Fig. 5.** This member has the maximum stress of the bottom- its size will be used up to the joint *F* and possibly *for the entire length* rd. The *net area required is 16 900/16 000 = 1.06 sq in, or the net of one angle is 0.53 sq in. One leg of the angle is riveted to the with ¾-in rivets which is assumed to cut out a section* ⅞ in by the

thickness of the angle. From Table XI, page 365, we find that
¼-in angle has an area of 1.06 sq in. The area to be deducted
one rivet-hole is $\frac{7}{8} \times \frac{1}{4} = \frac{7}{32} = 0.22$ sq in. This leaves for the
angle $1.06 - 0.22 = 0.84$ sq in, which is well above the required
is the smallest angle which can be used and as all the other te
have less stress than *AD*, the tension-members will be made un
out. With the exception of *FK*, many designers would use but
the web-members. While the net area is ample for the stresses
practice, as one angle produces a ONE-SIDED PULL on the gusset-

Member *AB*, Fig. 5. This piece has the maximum stress of
a compression of 20 200 lb and a transverse load of 2 000 lb.
EFFECT OF THE TWO LOADINGS in this case must be determine
quite different from that followed for wooden construction.
fiber-stress must not exceed that found from some column-fo

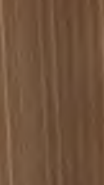


Fig. 6. Section through Rafter-member of Truss Shown in Fig. 5. (Axis at d_I from *AB*, through c. g. of angles; axis at *Cg*, through c. g. of sections; axis at d_{II} from *AB*, through c. g. of plate.)

example, $S_1 = 12\,500 - 500\ l/r$. The m
stress *S* may be found, approximately, fr
sion $S = P/A + Mc/I$. In this equati
direct compression, which is 20 200 lb
A the area of the section of the piece,
of inertia of this section, *c* the distance
most fiber of the section which is in con
its gravity-axis and *M* the maximum be
produced by the tranverse load. The
of the section must lie in the plane of
loading, if the above formula is use
metrical sections, as two angles back
beam, or a channel, the principal axes
SYMMETRY, and the values of *I* and
found from the properties of rolled sh
in Chapter X. The first trial-section
in Fig. 6, consisting of two 2½ by 2½
and one 10 by ¼-in plate. To find t
inertia, *I*, and the radius of gyration, *r*, the center of gravity of
found first. The distance *X* of the center of gravity from *AB* (
from Equation (2), page 295,

$$X = \frac{\text{area of plate} \times d_{II} + \text{area of angles} \times d_I}{\text{area of entire section}}$$

From the properties of angles, Table XII, page 367, the distance
to the center of gravity of the angle is $d = 0.72$ in. $I = 0.7$ and
two angles = 2.38 sq in. The plate does not usually extend to
angles, a clearance of from ⅛ to ¼ in being allowed. A clear
assumed.

angles (page 367) $2 \times 0.70 = 1.40$
ge 338), $A\,(d_1 - X)^2 = 2.38\,(2.32)^2 = 12.81$
ire section, $I = 47.22$

section, $I = Ar^2$ or $r = \sqrt{I/A}$, hence for this section $r = \sqrt{47.22/4.88}$ (See Equation (2), page 333.) The distance to the outermost fiber ion from the axis Cg is 3.04 in $= c$. There is now sufficient data to he actual fiber-stresses due to the loading and also the permissible e bending moment produced by the transverse load is

$$M = \tfrac{1}{8}\,(2\,000 \times 6.16 \times 12) = 18\,480 \text{ in lb}$$
$$S = 20\,200/4.88 + (18\,480 \times 3.04)/47.22 = 5\,330 \text{ lb per sq in}$$
$$S_1 = 12\,500 - 500 \times 6.16/3.11 = 11\,510 \text{ lb per sq in}$$

vs that the actual fiber-stress is very much smaller than the allowress, but as we have used minimum-size angles and a minimum thick- plate, the only way to reduce this section is to use a smaller plate. feasible because of the requirements for making proper connections s. The above analysis assumes that the member is prevented from ewise by the roof-covering. If such is not the case, r will have to ed for a vertical axis through the center of gravity of the section. g the moment of inertia,

e, $ht^3/12 = (10 \times 0.25^3)/12 = 0.026$
les, $2 \times 0.70 = 1.400$
$2.38\,(0.72 + 0.125)^2 = 1.699$
re section, $I = 3.125$

$$r = \sqrt{3.125/4.88} = 0.8 \text{ in}$$
$$S_1 = 12\,500 - 500 \times 6.16/0.8 = 8\,650 \text{ lb per sq in}$$

greater than the value of S, and hence this section fulfills all the re- considering the unsupported length vertically and sidewise as

***EF*, Fig. 5.** Taking two $2\tfrac{1}{2}$ by 2 by $\tfrac{1}{4}$-in angles with the $2\tfrac{1}{2}$-in back, the least value of $r = 0.78$ in (Table XVI, page 371).

$$S_1 = 12\,500 - 500 \times 12/0.78 = 4\,810 \text{ lb per sq in}$$
$$5\,000 \div 4\,810 = 1.04 \text{ sq in, required.}$$

of the two angles used is $2 \times 1.06 = 2.12$ sq in (Table XVI, page 371).

***CD*, Fig. 5.** The stress in this member is very small and one angle y fulfill the requirements. For one angle, $2\tfrac{1}{2}$ by 2 by $\tfrac{1}{4}$ in, the least Table XI, page 365) and $S_1 = 5\,300$ lb per sq in, indicating that this a large excess of strength. As pointed out above, it is better to use

ss-Ratio. The best specifications limit the ratio of the least dimen- nsupported length of a compression-member to 50, unless the allowess as given by the column-formula is decreased. The member *EF* p and about 144 in long, so that its length is 57.6 times its least As there is a great excess of area, the actual unit stress is much given by the formula.

s. The compression-members made up of two angles and designed in the preceding paragraphs, have been considered as if acting as It is clear that the various parts must be so fastened together that piece will buckle. If l is the unsupported length of the member

as a whole, r the corresponding least radius of gyration, r' th
gyration for any part and l' the unsupported length of the part
between stay-rivets, there is the following relation:

$$l'/r' = l/r \text{ or } l' = lr'/r$$

For the member *EF* $l' = 144 \times 0.42/0.78 = 78$ in.
Practice reduces this to 2 or 3 ft.

Tension-Members, also, should be riveted together in a si
make the parts pull together.

Example 5. The next truss considered is the Fink truss
in which two angles are used for all members and ¾-in rivet
tions.

Member *AC*, Fig. 7. For a unit stress of 16 000 lb per sq
required is 21 800/16 000 = 1.36 sq in. Two 2½ by 2½ by ¼-
section-area of 2.38 sq in (Table XII, page 367). Deducting
sq in, the net section becomes 1.94 sq in, while the required

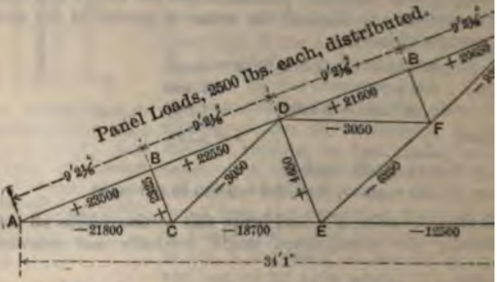


Fig. 7. Fink Truss. (See, also, Figs. 22, 22A, 22B, 22C

Members *CE* and *EH*, Fig. 7, are composed of angles of the

Members *CD*, *DF*, *EF* and *FG*, Fig. 7, are made of two
angles with a net area of 2.12 sq in. This greatly exceeds the

Members *BC*, *BF* and *DE*, Fig. 7. From the preceding exa
that a pair of minimum-size angles will be quite sufficient.
¼-in angles, having a section-area of 2.12 sq in, are used.

Member *AB*, Fig. 7. For this member in which there is
sion of 23 500 lb and a transverse stress due to a load of 2 500
trial is made with two 5 by 3½ by ⅜-in angles, with the 5-in legs
separated by a ¼-in gusset-plate. The moment of inertia abou
through the center of gravity of the two angles and parallel to
is (Table XI, page 363) 7.78* and the corresponding radius
in. About a vertical axis the radius of gyration is (Table
1.42 in, which is the least radius to be used in the colum

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* It will be noticed that the values given for the properties or element
in this example differ slightly from those given in the tables referred to,
I, r, x, etc. This changes the result very slightly and is due to varia
figures of values in different editions of manufactures' handbooks.

produced by the 2 500-lb load at the center of the member is ⅛ .2 × 12) = 34 500 in lb. The section-area of the two angles is (page q in.

$$S = 23\,500/6.10 + (34\,500 \times 1.61)/7.78 = 10\,990 \text{ lb per sq in}$$
$$S_1 = 12\,500 - (500 \times 9.2)/1.42 = 9\,260 \text{ lb per sq in}$$

s less than S, it is seen that the angles selected are a little too light. using angles of greater thickness it will be better to select a larger wo 6 by 3½ by ⅜-in angles are used (page 363)

$$S = 23\,500/6.86 + (34\,500 \times 2.04)/12.86 = 8\,890 \text{ lb per sq in}$$
$$S_1 = 12\,500 - (500 \times 9.2)/1.34 = 9\,070 \text{ lb per sq in}$$

s that there is ample strength and stiffness and that the area is in- 0.76 sq in. If two 5 by 3½ by 7/16-in angles had been used, the area e been increased 0.96 sq in (page 363). The least radius of gyration e expression for S_1 assumes that the angles will be separated by ¼-in tes. If thicker gusset-plates are used, the value of r will increase.

Details. The use of UNIFORM SIZES for members in the same straight nomical and adds rigidity to the truss. The angles can be furnished ths of 60 ft and over, thereby reducing the labor of cutting them and the number of rivets and the size of the gusset-plates. The portion s *AEG* shown by Fig. 7 would be completely riveted up in the shops, ly three joints to be riveted at the building. In general, any truss one outside dimension not exceeding 10 ft, can be shipped by rail. rns the location of the splices.

3. Joints of Wooden Trusses

ints of any truss should be proportioned with as much care as is used ning the sizes of the members, so that the truss will be equally strong parts. The general principles and methods for designing joints are in Chapter XII and illustrated by examples. To further explain the he methods of design of some of the joints for the trusses shown in d 3 are added in this chapter.

Fig. 1. This is the most important joint in the truss. There are ns for this joint, but only a few of them are illustrated. Fig. 8 shows a LTED JOINT. The rafter rests in a notch in the bottom chord and is ace by one or more rolled-steel bolts. These bolts are perpendicular s of the rafter, and the stresses in them are found graphically by the *bc* (Fig. 8) in which *ac* is perpendicular to the SCARF-CUT or SEAT of . The tension in the bolts is found to be 31 250 lb, and with a permis- s of 16 000 lb per sq in, the net section-area required is 1.95 sq in, responds to one 1⅞-in bolt (Table II, page 388). The WASHER, bear- the grain of the rafter, will have an area of 31 250/200 = 156 sq in 8). Since the top-chord is actually but 5½ in wide, the length of the bout 28 in. Such a plate would look out of proportion with one bolt, in bolts are substituted, having a net section-area of 2.10 sq in (Table 388). Two bolts are placed near each end of the plate and one bolt in the middle. The bolts are spaced about 9½ in apart. The thick- e plate may be taken as one-fifth the distance from the end of the he nuts of the first pair of bolts. This distance is about 3.4 in; hence ess is 0.67 in. A *¾-in plate is used. The lower end of each pair of ovided with a PLATE-WASHER bearing upon the inclined surface of the BOLSTER as shown. The ANGLE OF INCLINATION* approximates 45°

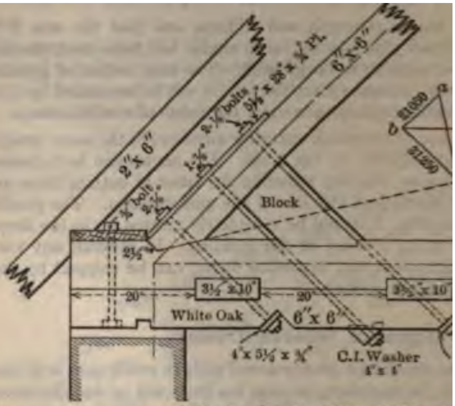
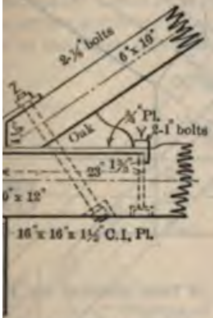


Fig. 8. Detail of Joint 1, Fig. 1

lb, and taking the actual thickness of 6 in to be 5½ in
the key will safely carry 5½ × 200 = 1 100 lb in long
page 412). The keys will, therefore, be 10 in long. Th
against the notch in the white-pine chord, and for e
depth of the notch carries 5½ × 1 100 = 6 050 lb. T
therefore, is 1.8 in in the chord. In the bolster each
carries 5½ × 1 400 = 7 700 lb (Table XVI, page 647),
is 1.43 in. This makes the total thickness of the keys
say 3½ in. The size of the keys is 3½ by 5½ by 10 in
KEYS is governed by the longitudinal shear of the w
inch in length carries 5½ × 100 = 550 lb (Table I, pa
distance between keys is 11 050/550 = 20 in. The v

TONE OF METAL PLATE provided to distribute the pressure. (See
In this case a 16 by 14 by 1¼-in CAST-IRON PLATE is used,
he pressure on the brickwork to 82½ lb per sq in.

3. This joint might be made in the manner described above,
nown in Fig. 9 is used. The thickness of the plate is usually
e thickness required at Y to give the HOOK the proper strength.



il of Joint 1, Fig. 3

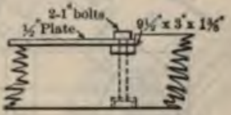


Fig. 9A. Alternate Detail of Joint 1, Fig. 3

tically takes one-half the horizontal component of the stress in
h is the stress in the bottom chord in this case) as the bolts are
to keep the parts in place. The metal bears against the end-
inch in depth of the notch, the fibers carry $9\frac{1}{2} \times 1\,100 = 10\,450$ lb,
otch is $\frac{1}{2}\,(27\,350/10\,450) = 1.31$ in deep, say 1⅜ in. Considering
WROUGHT-IRON CANTILEVER, 1⅜ in long and uniformly loaded
er sq in, the thick-
om the expression
$\frac{1}{6}(12\,000 \times 1 \times t^2)$,
n. The nearest
is a thickness of
ngth of the bot-
ecessary to take
om the hook in
ear is $9\frac{1}{2} \times 100$
in, or, $13\,675/950$
. The inclination
H with the ver-
out 36°, and the
re on this surface
$200)\,(^{36}/_{90})^2 = 344$
$0/(344 \times 9\frac{1}{2}) = 8.3$ in, which is the required depth of the cut.
re confined by the plate, one-half this value, or 4¼ in, approx-
l. The bolts, Z, are two ⅞-in bolts. There should be two
fully placed so that the hook bears against them. There is a
for the hook to straighten and hence 1-in bolts are used.
of the plate in tension is evidently greatly in excess of that

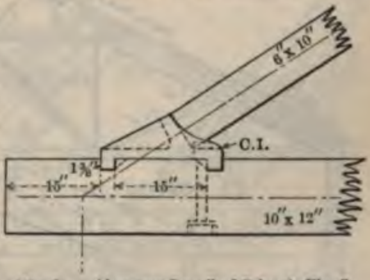


Fig. 9B. Alternate Detail of Joint 1, Fig. 3

3. A better detail at Y, Fig. 9, is shown in Fig. 9A. It is
re that one-half the tension is *taken by the notch* at Y. The

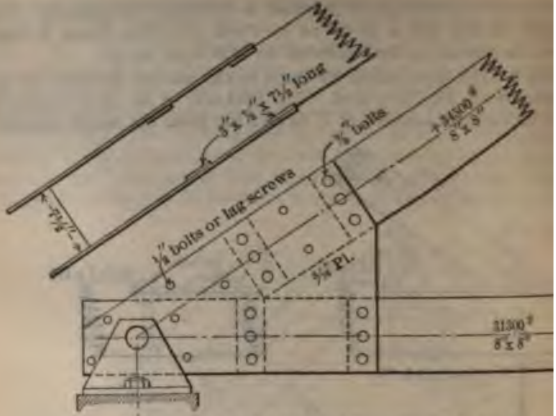


Fig. 9c. Detail of Joint 1, of Truss Similar to Fig. 1

depth of the notch is 1⅜ in and the size of the metal block, 1⅜ by
The bolts are assumed to have a close fit in the block and in the plate

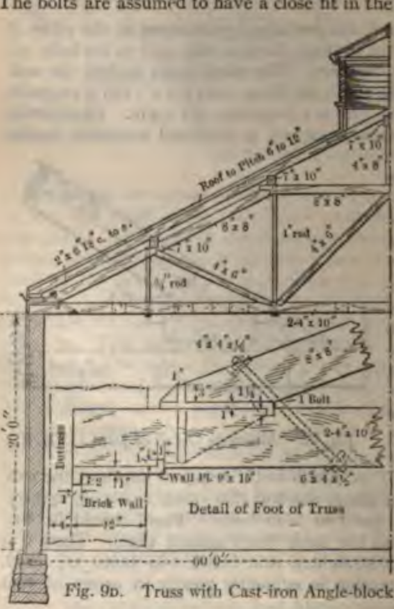


Fig. 9D. Truss with Cast-iron Angle-block

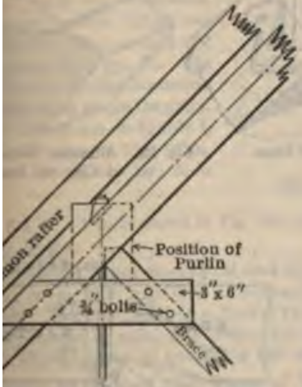
carry the stress in s
At 10 000 lb p
the area of the
(27 350/10 000) =
requiring two 1-in
(Table III, page
thickness of the
necessary to give
bearing against th
½ (13 675) ÷ 20 000
.34 in. A ½-in pla
fore ample.

Joint 1, Fig. 3.
CAST-IRON ANGLE-
be used in this par
as shown in Fig. 9

Other Details f
Fig. 3. Another
this joint, but f
truss, is shown i
The rafter and bo
are of long-leaf
and the metal pa
The stresses are
through 3 by ¾-i
bearing against th
of the wood, and
plates to the s
through the bolts in bending. The side plates should be drawn
the wood by LAG-SCREWS, as shown, to prevent buckling when in

hows a good application of the CAST-IRON ANGLE-BLOCK used in the blacksmith-shop of the Boston & Maine Railroad Company. The d shearing values are provided for principally by a tenon on the to the bottom chord as indicated by the dotted lines.

Fig. 1. Where a brace abuts against a rafter, as in this joint, one cut of the brace should bisect the angle made between the brace and the the second cut should be at right angles to this, as shown in Fig. 10. then set in a notch or mortise to keep the brace in place and to transssure to the rafter. The purlin may be supported by a 3-in plank, as



Detail of Joint 2, Fig. 1, with Rod Added

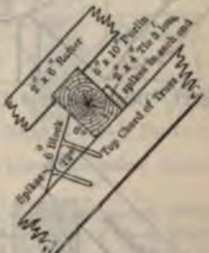
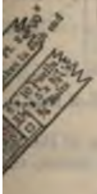


Fig. 10A. Purlin-connection. Purlins on Top of Truss-chord



Purlin-con with Steel-

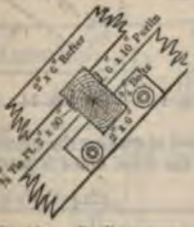


Fig. 10C. Purlin-connection with Wooden Bearing-block

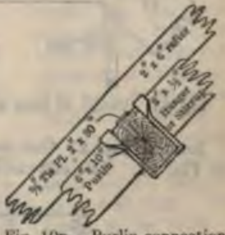


Fig. 10D. Purlin-connection with Beam-hanger

Fig. 10. Some form of METAL HANGER, of the DUPLEX TYPE is rred. In the truss shown in Fig. 1, there is no vertical rod at this many trusses have a rod there, and one is therefore shown in Fig. 10. r on top of the rafter must have sufficient area to transmit the stress to the rafter. Other forms of purlin-connections are shown in Figs.

King-Rod Truss. Fig. 11 shows the joint at the top of a KING- with a DUPLEX HANGER to support the purlin. The wrought-iron ate for large trusses should extend along the top of each rafter a istance to permit its being fastened by LAG-SCREWS or BOLTS. Fig. 12 STING in place of the ROLLED PLATE.

Joint 3, Fig. 1. This should be made as shown in Fig. 13. Th
cuts bisect the angle made between the two 6 by 6-in pieces. In pl
CAST-IRON WASHER a WROUGHT-IRON or STEEL PLATE may be used.

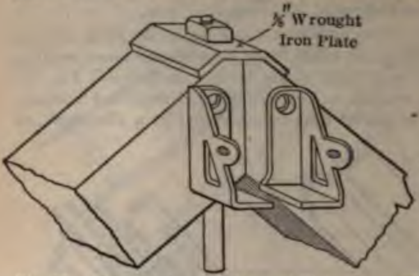


Fig. 11. Detail of Apex of King-rod Truss

Fig. 12. Alternate Detail
of King-rod Trus

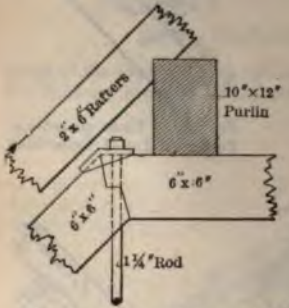


Fig. 13. Detail of Joint 3, Fig. 1

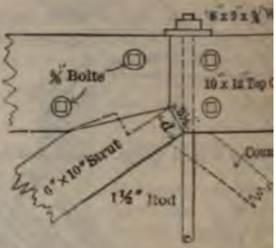


Fig. 14. Detail of Joint 2, F

Joint 2, Fig. 3. One method of making the connections at this joi
in Fig. 14. The end-cut of the main brace is made as shown, the

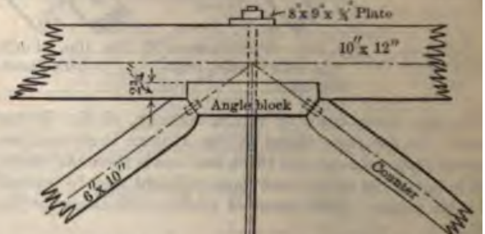


Fig. 15. Alternate Detail of Joint 2, Fig. 3

being determined by the necessary area of the inclined cut in the
The permissible unit pressure is about 525 lb per sq in. Then 33 450
64 sq in, or the distance d is a little greater than the depth of the br

il can only be used for the end-brace by making two notches as e dotted lines. A much better method is shown in Fig. 15, where ock is used. The angle-block is made of very hard wood so that of the brace is provided for, and it is notched into the chord a suffit to transfer the horizontal component of the stress in the brace . A notch 1 in deep carries 1 100 × 9½ = 10 450 lb (Table I, page e for a horizontal component of 27 350 lb (Fig. 4), the notch is n deep. This clearly races should be inclined ' with the horizontal, ard or weak details are ted. The vertical rod ress of 13 492 lb. The op of the chord transfers bearing across the grain. ress of 200 lb (Table I, the area is 67.4 sq in, 8 by 9 by ¾-in

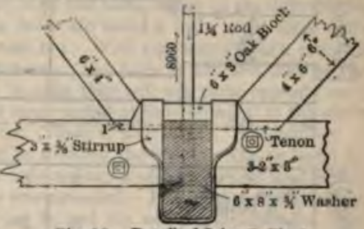
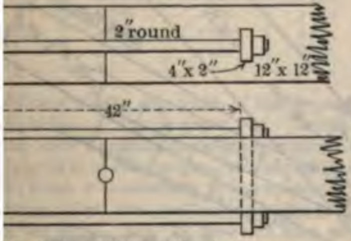


Fig. 16. Detail of Joint 7, Fig 1

g. 1. This is shown in Fig. 16, and the above discussions cover all design.

Since it is not economical and often impossible to procure timbers or 30 ft in length, it is necessary to make one or more SPLICES . The top-chord of a HOWE TRUSS is spliced by placing the timnd, and by spiking or bolting on side planks to keep them in place. chord cannot be treated in this manner, as it is in tension.

ice or Tabled Fish-Plate of Wood. It is assumed that the of the truss shown in Fig. 3 is to be spliced at the middle of the span. Fig. 17A shows this splice. It is assumed that the side pieces are of white pine. The total depth of the notches is 48 560 ÷ (1 100 × 11½) = d = 3.84 in (Table I, page 1138). Each notch, then, is about 2 in deep. The length of the table is l = ½ [48 560 ÷ (100 × 11½)] = 21 in. The net thickness of each side 8 560)/(700 × 11½) = 3 in, without deducting anything for the two The chord-pieces have less than the required area because of the required; hence a 12 × 12-in timber is required if this form of . The proper dimensions are shown in Fig. 17A.



7. Splice of Bottom Chord of Truss

ice. Fig. 17 shows an old and very efficient form of splice, proreplace the form shown in Fig. 17A.

r Built-up Chord. The top chord, when BUILT UP of 2-in planks, ough spiking with two ¾-in bolts at the ends of each plank. The l, which is in tension, should be so arranged that the ends of one strand are well removed from the *ends in other strands.* The

middle strand of a BUILT-UP CHORD is completely cut away to permit th
of the vertical rods. The strands should be thoroughly spiked, and bol
2 ft, care being taken to see that the bolts do not come nearer th
from the end of any plank. While BUILT-UP MEMBERS are in favor wi

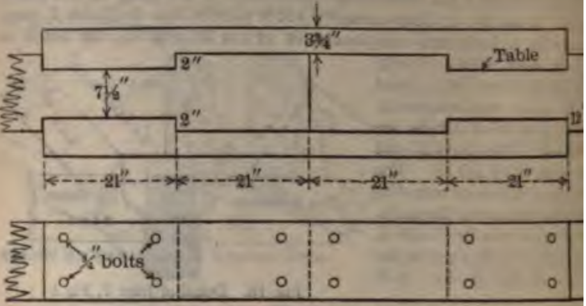


Fig. 17A. Alternate Detail for Splice of Bottom Chord

ers because the materials are readily obtained, yet for important stru
writer believes it is worth while to use a little more effort and pay a li
to get SOLID STICKS for truss-members.

Wall-Joint of Scissors Trusses. In SCISSORS TRUSSES the joint
wall formed by the rafter and tie-beam should always be carefully pro

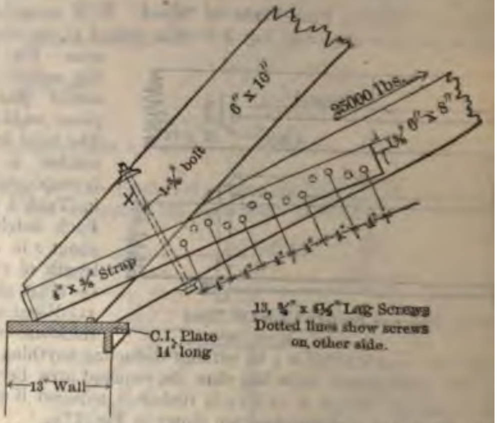


Fig. 18. Wall-joints for Scissors Trusses, Figs. 24 to 27, Chapter X

to the stresses; otherwise the joint is liable to open and the wall to
out. Much greater strength is required in this joint than in the wall
KING-ROD TRUSS of the same span, because the stresses in a SCISSORS
usually at least twice and sometimes three or four times as great as
with a horizontal tie-beam. For a SCISSORS TRUSS built of planks, as

rough the center of each joint, with as many spikes as can be linarily give sufficient strength. For trusses like those shown in of Chapter XXVI, one of the best methods of making the wall-ne roof is quite flat, is that shown in Fig. 18, which is the detail int where the stress in the tie-beam was 25 000 lb. It should be he WROUGHT-IRON STRAP is secured to the tie by LAG-SCREWS in-. It is practically impossible to bolt a strap to each side of a get a good bearing for all of the bolts, owing to the difficulty in es straight; and if the holes are bored a little large, some bolts the wood and some may not. With LAG-SCREWS each screw is a good bearing in the wood. The holes in the two sides of the course, be staggered, so that they will not come opposite each et sectional area of the strap should at least be equal to the stress n divided by 2 × 12 000 (Table I, page 1138). The number of or both sides, is found by dividing the stress in the tie-beam by of one screw. For the safe resistance of LAG-SCREWS used in this s given in Table V are recommended. In the joint shown in Fig. in the tie-beam is 25 000 lb, and the wood is Douglas fir. The ierefore, require a sectional area in the strap of ½ (25 000)/12 000 nd twelve ¾-in lag-screws.

Safe Resistance of Lag-Screws When Used as in Fig. 18

in	Safe resistance in pounds				Minimum thickness of strap in inches
gth	Oak	White pine	Douglas fir	Long-leaf pine	
½	800	600	700	800	¼
	1 400	1 000	1 100	1 200	¼
	2 000	1 500	1 650	1 800	5/16
½	2 500	1 800	2 100	2 400	5/16
	3 000	2 400	2 800	3 000	⅜

kness of ⅜ in, the width of the strap necessary to give a sectional in is 1.05/.375, or about 3 in. To this should be added the diam-g-screw to obtain the working width. Thus 3 + ¾ = 3¾ in. The by ⅜ in in cross-section, as some additional strength is obtained by which it is necessary to insert to hold the timbers together while eing raised into position, and also to bring them tightly together the strap. Fig. 19 shows another method of making this joint e used with advantage when the inclination of the rafter is less he advantage in using this truss is that if it is erected ONE PIECE e tie-beams may be put up first, thus providing a SEAT to receive The strap prevents the end of the rafter from springing up. The e bolt should be proportioned to the horizontal component of the after. Fig. 20 shows a good form of joint to use at joint 5 of Fig. XXVI, when it is desired to substitute a wooden tie for the rods 27. The sectional area of the strap and the number of lag-screws portioned by the rules given for Fig. 18.

Where iron or steel rods are used in wooden trusses, washers are ler the heads and nuts to properly distribute the loads on the limensions of the washers are determined by the *allowable* bearing pressure on the wood and the magnitudes of the loads. Table VI gives th allowable loads which can be transmitted by standard round cast washers rectangular washers bearing across the wood fibers. Table VII gives the di- mension of standard round cast washers. The bearing areas of these wash

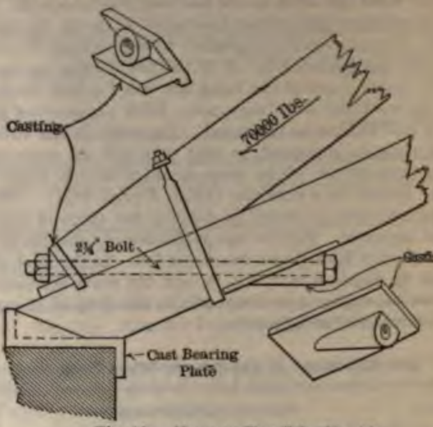


Fig. 19. Alternate Detail for Fig. 18

are too small for use on the softer woods and, therefore, except when the are small, it is better to use rectangular washers of iron or steel plate. large washers should be cast, and should have the form shown in Fig. The use of the ribs gives the required strength and saves considerable mater

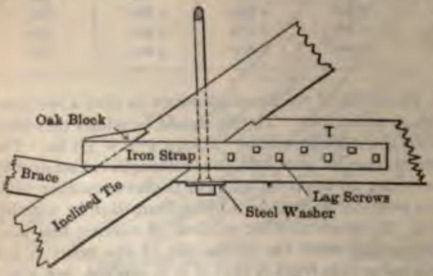


Fig. 20. Detail of Joint 5, Fig. 27, Chapter XXVI

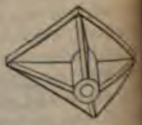


Fig. 20A. Cast Washer with Brack

Thickness of Rectangular Steel-Plate Washers. The thickness tangular steel-plate washers can be found from the following formulas in wh l is the distance from the edge of the plate to the nut and t the thickness of plate. When used

On white oak.................................... $l = 3.4t$
On white pine.................................... $l = 3.2t$
On long-leaf yellow pine.......................... $l = 3.9t$
On short-leaf yellow pine......................... $l = 4.6t$

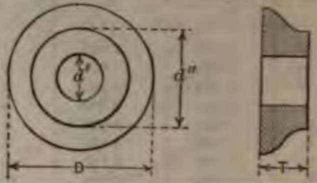
VI. Safe Bearing Resistance of Cast-Iron Washers, in Pounds

	Area,* sq in	White pine, lb	Short-leaf yellow pine, lb	Long-leaf yellow pine, lb	White oak, lb
			Round washers		
	5.16	1 030	1 290	1 810	2 580
	6.69	1 340	1 670	2 340	3 350
	7.78	1 560	1 950	2 720	3 890
	10.4	2 080	2 600	3 640	5 200
	11.7	2 340	2 930	4 100	5 850
	16.6	3 320	4 150	5 810	8 300
	26.9	5 380	6 730	9 420	13 500
	28.6	5 720	7 150	10 000	14 300
	38.5	7 700	9 630	13 500	19 300
	49.9	9 980	12 500	17 500	25 000
	62.8	12 600	15 700	22 000	31 400
	77.1	15 400	19 300	27 000	38 600
	92.9	18 600	23 200	32 500	46 500
	110.2	22 000	27 600	38 600	55 100
			Rectangular washers		
	24	4 800	6 000	8 400	12 000
	32	6 400	8 000	11 200	16 000
	36	7 200	9 000	12 600	18 000
	42	8 400	10 500	14 700	21 000
	48	9 600	12 000	16 800	24 000
	54	10 800	13 500	18 900	27 000
	60	12 000	15 000	21 000	30 000
	64	12 800	16 000	22 400	32 000
	72	14 400	18 000	25 200	36 000
	80	16 000	20 000	28 000	40 000
	96	19 200	24 000	33 600	48 000
	100	20 000	25 000	35 000	50 000
	110	22 000	27 500	38 500	55 000
	120	24 000	30 000	42 000	60 000
	140	28 000	35 000	49 000	70 000
	144	28 800	36 000	50 400	72 000
	168	33 600	42 000	58 800	84 000
	192	38 400	48 000	67 200	96 000
	196	39 200	49 000	68 600	98 000
	224	44 800	56 000	78 400	112 000
ues,	lb per sq in	200	250	350	500

ctual areas bearing on the wood are given for round washers. For rectangular e total area is given, no allowance being made for holes.

. Many other forms of connections are in use and their proper design mands that the methods explained in Chapter XII and in this chapter ently followed. All details are not suitable for all cases and the de-st use common sense in the selection of the PARTICULAR TYPE to be n its design. Wood is very variable in its properties and consequently TORS OF SAFETY are used for certain kinds of stress and smaller factors for others. Heavy trusses, in which the sizes of the members are sel cording to the magnitudes of the stresses, should be very carefully wo in every detail, while small trusses with large excess of material do not as much care.

Table VII. Proportions of Standard Cast-Iron Washers



Diam of bolt, *d* in	*D* in	*d''* in	*d'* in	*T* in	Weight, lb
½	2⅝	1¾	9/16	⅝	½
⅝	3	1⅞	11/16	¾	¾
¾	3¼	2⅛	13/16	⅞	1¼
⅞	3¾	2½	15/16	⅞	1½
1	4	2¾	1 1/16	1⅛	2½
1⅛	4¾	2¾	1 3/16	1⅛	3
1¼	6	3	1 5/16	1¼	5¾
1½	6¼	3¼	1⅝	1½	6
1¾	7¼	3¾	1⅞	1¾	9½
2	8¼	4¼	2⅛	2	17¼
2¼	9¼	4¾	2⅜	2¼	20
2½	10¼	5¼	2⅝	2½	27¼
2¾	11¼	5¾	2⅞	2¾	36
3	12¼	6¼	3⅛	3	46

For sizes not given, $D = 4d + \frac{1}{4}''$ $d'' = 2d + \frac{1}{4}$
$d' = d + \frac{1}{8}$ $T = d$

4. Joints of Steel Trusses

Trusses with Riveted Joints are usually made with ANGLES for members and generally for the chords, although the latter are sometir of a pair of CHANNELS or of two ANGLES and a WEB-PLATE. The men connected at the joints by means of GUSSET-PLATES, to which all of the are RIVETED. Typical examples of riveted joints in roof-trusses are Figs. 22 to 24E. When the rafter or chord has a WEB-PLATE, as in the web-members are riveted to this plate and a GUSSET-PLATE is not except at the end-joint and apex, as shown in Figs. 23A and 23E. In o there shall be no twisting, it is necessary to make the principal membe truss DOUBLE, so that the gusset-plates can be riveted between them single angles are used for web-members and two such members com joint they should be riveted to opposite sides of the gusset-plates. F strength the thickness of the gusset-plate should be such that the BE the rivets equals the strength of the rivets in DOUBLE SHEAR, the thickn ever, not exceeding the combined thickness of the two angles. Pract siderations seldom make the gusset over ⅜ in thick for ordinary con

hout serious
ass through
of gravity
tions. The
rivets re-
each mem-

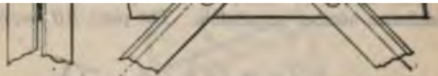
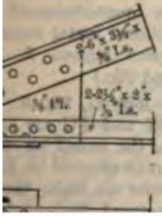


Fig. 21. Riveted Truss-joint with Truss-diagram Lines

e determined according to the stress in that member, the resistance
ts being considered for both SHEARING and BEARING. The method
ning the number of rivets in a joint is explained in Chapter XII,
w more clearly the application to truss-joints, the joints for the
in Fig. 7 will be designed.

Considerations, Truss of Fig. 7. It is assumed that the truss
ped in three parts, making all the joints SHOP-RIVETED except those
he splices at each end of the piece *EH*. All gusset-plates are to be
and all rivets ¾-in, except in the 2-in legs of angles, where ⅝-in
to be used. Since the bearing of a ⅜-in plate on a ¾-in rivet at
er sq in (Table I, page 1138) is 5 630 lb, or at 18 000 lb per sq in
, page 419) is 5 060 lb, and the resistance of the rivet in double shear,
=8 840 lb (Table III, page 419), the number of rivets in all the joints
by the bearing value. Only one leg of the angles will be connected



stress. In like manner four rivets are required for the bottom supporting force or the reaction is transferred to the gusset through chord prolonged. In this case the reaction is about 8 800 lb which rivets. Fig. 22 shows the arrangement of this joint at the expansi

Joint D, Fig. 7. The web-members each require less than one ri or three should be used. Since the top-chord angle is continuous, th rivets in it is determined by the difference between the two adja and the load of the purlin if it rests on the chord. Here again the rivets required falls below the minimum number. Fig. 22A shows t

Joint E, Fig. 7. The piece *CE* requires four rivets and the web- minimum number permissible. The piece *EH* requires, at 20 000 l

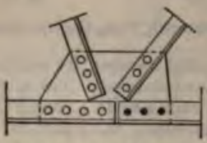


Fig. 22B. Detail of Joint *E*, Fink Truss, Fig. 7

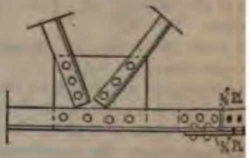


Fig. 22C. Detail of Joint *E* and S *EH*, Fink Truss, Fig. 7

bearing value, 12 500/5 630 = 2.22 rivets; but as this connection i made IN THE FIELD, it is customary to increase the number 25%. T the required number three. Sometimes the outstanding legs are spl member *CE* by a plate. Without doubt this increases the strength o but it is doubtful if the increase in strength is enough to offset the Fowler's specifications do not permit the piece *EH* to be connect gusset-plate. They specify that the connection shall be made upon t *E*. This arrangement allows the use of a smaller gusset-plate at *E* be counterbalanced b ditional metal requir splice beyond *E*. (and 22C.)

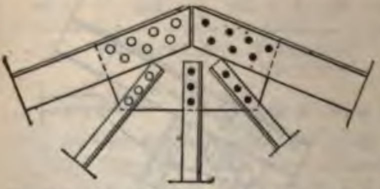


Fig. 22D. Detail of Joint *G*, Fink Truss, Fig. 7

Joint G, Fig. 7. *BG* and *FG* are sh to the gusset on on FIELD-RIVETED on the order to make the metrical, the numbe RIVETS is made the required for the FIELD-CONNECTION. In this case the top chord r rivets and the web-member three. Two rivets may be used in (Fig. 22D.)

Field-Connections. Bolts are often used instead of rivets for ma CONNECTIONS. If the bolts fit the holes snugly, there is no serious their use. In fact a good bolt is better than a poor rivet. For imp however, bolts should not be used unless turned true to size and dri holes. Open holes or holes for FIELD-RIVETS are indicated by BLAC

Shop-Drawings. It is not advisable for the architect to ma drawings for the steelwork. He should make what are usually d GENERAL DRAWINGS. These are made to scale and give the general

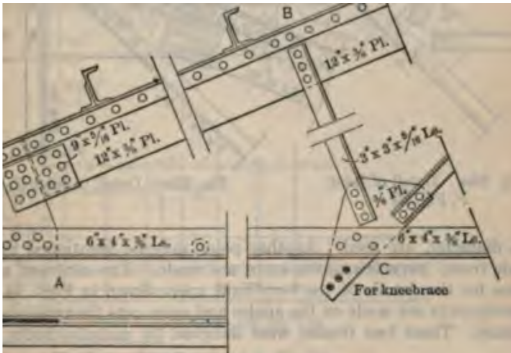


Fig. 23A. Detail of Joints *A*, *B* and *C* of Fig. 23

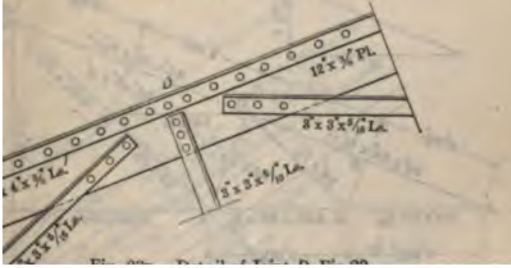


Fig. 23B. Detail of Joint D, Fig. 23

and sizes of the various members, and show each rivet approximately i proper position. The manufacturer who fabricates the structure prefers make his own SHOP-DRAWINGS to conform with his standards and methods.

Examples from Practice. Figs. 23 to 23E show the details of a mod shop-truss. These details were taken from the SHOP-DRAWINGS but with rivet-spacing omitted. No metal under 5/16 in thickness, or rivets under

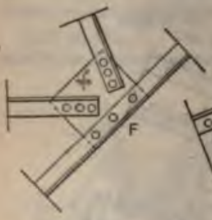


Fig. 23D. Detail of Joint *F*, Fig. 23

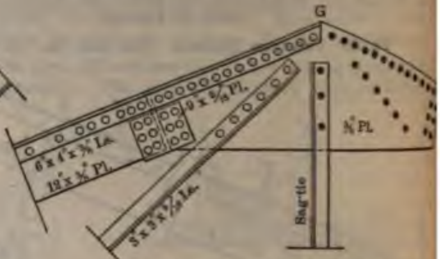


Fig. 23E. Detail of Joint *G*, Fig. 23

in diameter, are used. Another point may be mentioned in connection w this truss; very few BEVEL-CUTS are made. The contrary appears to be t case for the details of the very light truss shown in Figs. 24 to 24F, in wh BEVEL-CUTS are made on the angles and more cuts than necessary on the gus plates. These two trusses were designed by manufacturers widely separ

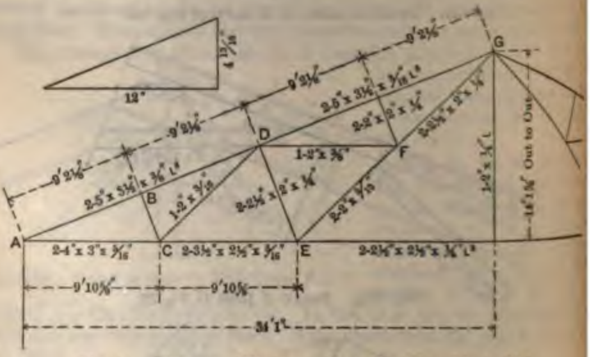


Fig. 24. Fink-truss Diagram. (See, also, Figs. 24A to 24F)

and are quite different in their details; and the variations emphasize the that the architect should not attempt to make SHOP-DRAWINGS.

Trusses with Knee-Braces. Fig. 25 represents joint 1 of Fig. 55, Chap XXVI, and was engraved from the WORKING DRAWING made by the New Jer Steel and Iron Company, Trenton, N. J. Another detail of a truss-conn to a column is shown in Fig. 26. This was used in the template-shop roof-tr Ambridge plant of the American Bridge Company, Ambridge, Pa. Fig

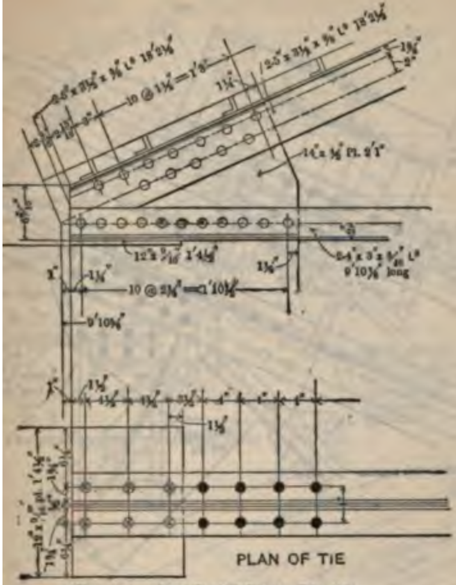


Fig. 24A. Detail of Joint *A*, Fig. 24

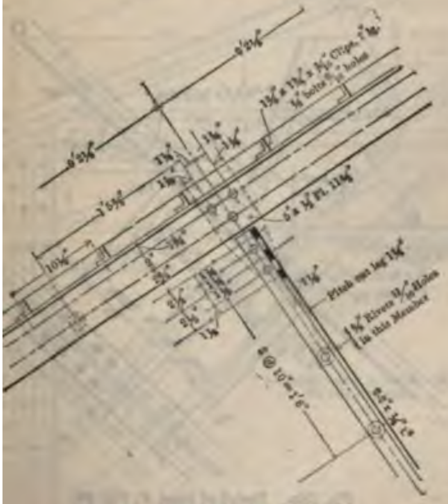


Fig. 24B. Detail of *Joint B, Fig. 24.*

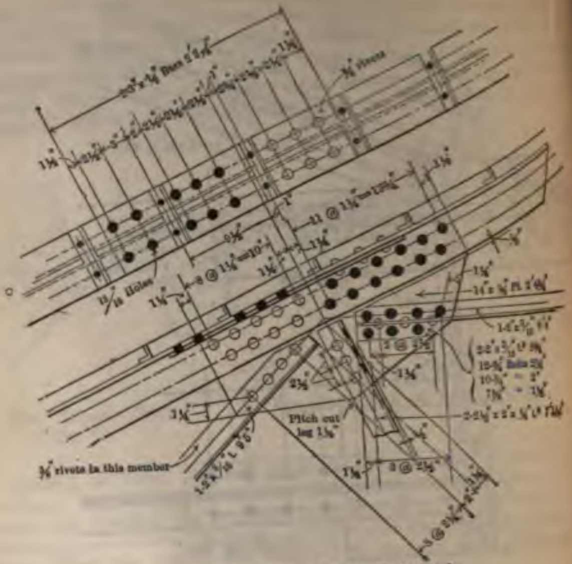

$\frac{3}{4}$" rivets
$\frac{13}{16}$ Holes
11 @ 1$\frac{3}{4}$" = 1'0$\frac{1}{4}$"
8 @ 1$\frac{1}{4}$" = 10"
1$\frac{3}{8}$"
2$\frac{1}{2}$"
Pitch out leg 1$\frac{1}{4}$"
$\frac{5}{8}$" rivets in this member
3 @ 2$\frac{1}{2}$"

Fig. 24c. Detail of Joint *D*, Fig. 24

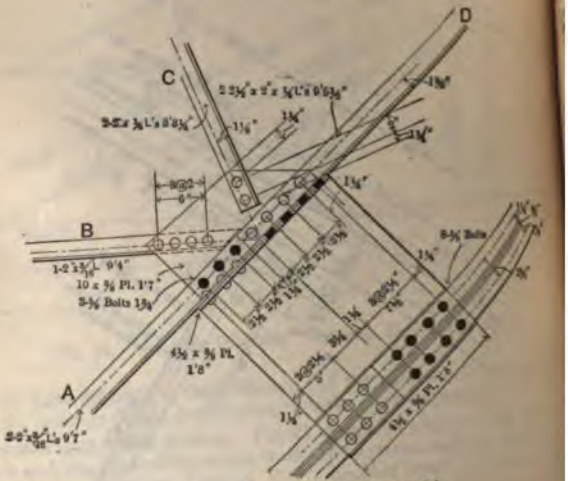

D
C
B
A
1$\frac{3}{8}$"
1$\frac{1}{8}$"
1$\frac{1}{4}$"
10 x $\frac{3}{8}$ Pl. 1'7"
4$\frac{1}{2}$ x $\frac{3}{8}$ Pl. 1'8"
4$\frac{1}{2}$ x $\frac{3}{8}$ Pl. 1'8"

Fig. 24d. Detail of Joint *F*, Fig. 24

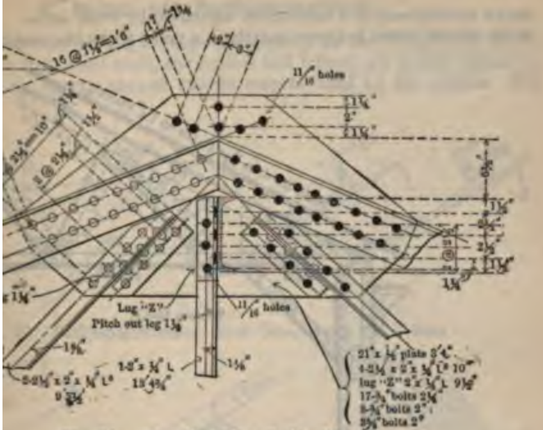
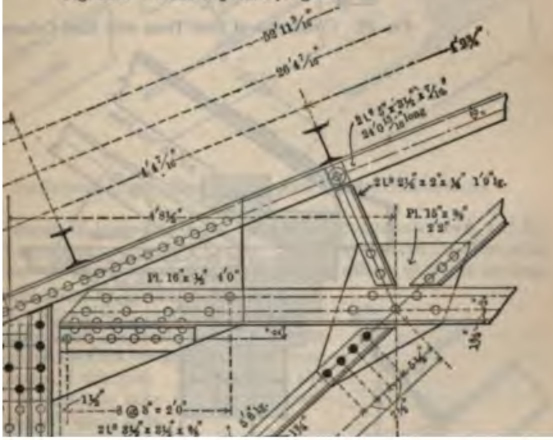


Fig. 24E. Detail of Joint G, Fig. 24



shows the wall-end of a small truss supported by brick walls. The
of the STRESS-LINES is approximately in a point above the center of

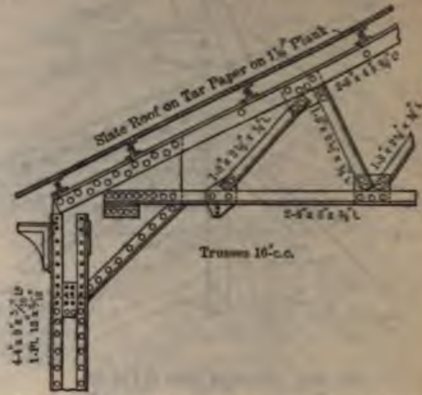


Fig. 26. Connection of Steel Truss with Steel Columns

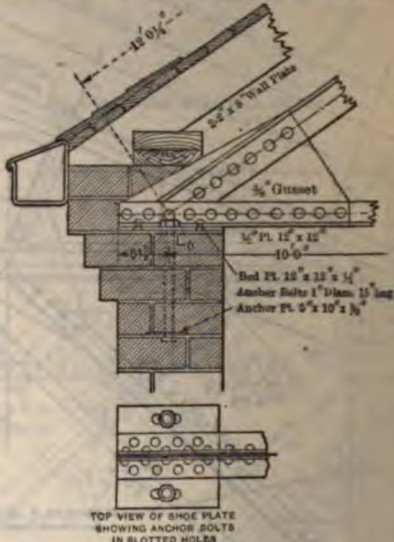
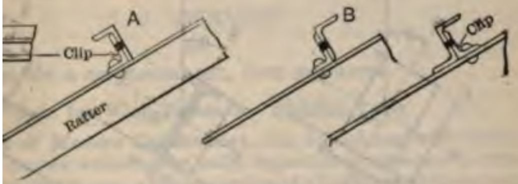


Fig. 27. Wall-end of Steel Truss. Support and Anchoring-

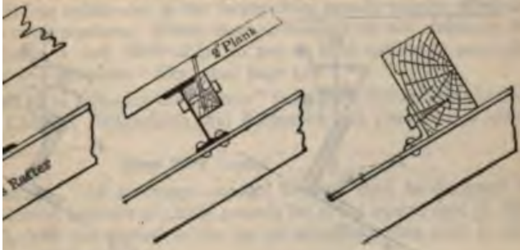
This condition is seldom considered by architects. Usually it is
any extra expense to satisfy this requirement and thereby to a great
vent unknown bending-stresses.

5. Purlins and Purlin-Connections

re the roofing is supported directly on the PURLINS, as is gen- light steel roofs, the purlins and trusses are usually spaced so at SIMPLE ROLLED SHAPES may be used for the purlins. For



8. Purlin-connections. Steel Clips, Angles and Z Bars



urlin-connections. Steel Sections with Wooden Nailing-strips

usses of from 8 to 10 ft, ANGLES are commonly used, and for ARS, CHANNELS and I BEAMS. WOODEN PURLINS are often used . If STEEL PURLINS support wooden rafters or plank roofing, of wood is bolted to the purlin, as When the distance between purlins a line of $\frac{5}{8}$-in rods should run from the purlins, to prevent them from ane of the roof. The purlin at the esigned to take the vertical com- ess in these rods.

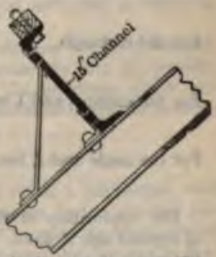


Fig. 30. Purlin-connection. Braced Channel

ctions. Figs. 28, 29 and 30 show a methods of fastening the purlins to

rlins. Fig. 31 shows the cross- CTANGULAR WOODEN PURLIN and of STEEL SHAPES employed for purlins. , when considering wooden purlins he stress in the outer fiber is true with reference to the PRINCIPAL AXES of the section. Then, axis does not lie in the plane of the loading, the loading d into two components, respectively, parallel to the two

Let f' = the fiber-stress with reference to the principal axis, AA, for the
tangle, 1–1 for the **I** beam and channel, and 4–4 for the angle and Z bar. M'
the bending moment of the component of the load which lies in the plane
pendicular to the above axis. I' = the moment of inertia of the section

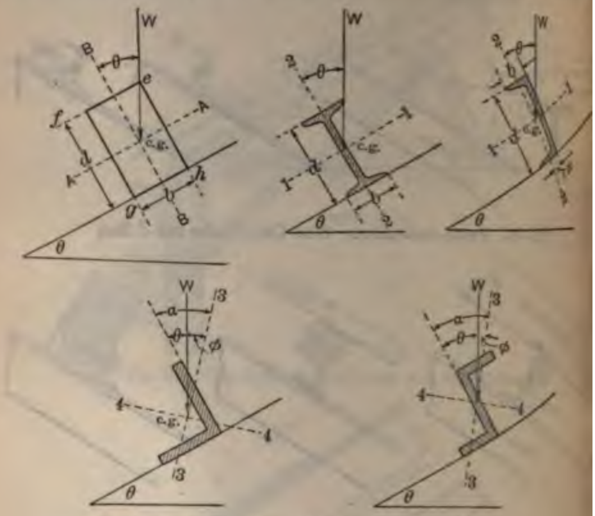


Fig. 31. Sections of Wooden and Steel Purlins

reference to the above axis. c' = the distance of any selected fiber from the
axis. For the other principal axis use, f'', M'', I'' and C''; then if f is the
sultant fiber-stress,

$$f = f' + f'' = M'c'/I' + M''c''/I''$$

For the rectangle,

$$f = f' + f'' = 6\,M'/bd^2 + 6\,M''/b^2d$$

For the channel and **I** beam,

$$f = f' + f'' = M'd/2\,I_{1-1} + M''b/2\,I_{2-2}$$

For the angle and Z bar,

$$f = f' + f'' = M'c'/I_{4-4} + M''c''I_{3-3}$$

The application of these formulas offers no difficulties except in the
of ANGLES and Z BARS. For the other forms, the values of I and c are
the tables of properties of the sections (Chapter X). The locations of the
cipal axes for the Z bars and angles are also given in the tables, but the
of c are not given for any of the fibers. The easiest way to get the
in any particular case is to draw the section of the angle or Z bar full size
the principal axes and then measure the actual distances, c.

CHAPTER XXIX

-BRACING OF TALL BUILDINGS

BY
N. A. RICHARDS
OF
RDY & HENDERSON, INC., CIVIL ENGINEERS

ata for Wind-Pressure. Building Laws

of Modern Construction, that is, buildings with skeleton
urtain walls or filler walls, require that resistance to wind-
ed with care. The proportions of a building and the arrange-
of the walls determine to what extent special bracing must
ding ordinances in the larger cities usually require consider-
ind-pressure. Where such ordinances do not definitely fix
ure, a unit force of 30 lb per sq ft of surface is generally
nd adequate. (See, also, page 150.)

The following are extracts * from the building ordinances of
hicago, Philadelphia and Baltimore with reference to wind-

New York (1914)

ES. "All structures exposed to wind shall be designed to
ind pressure of thirty pounds for every square foot of sur-
from the ground to the top of same, including roof, in any

no case shall the overturning moment due to wind pressure
per centum of the moment of stability of the structure.
UCED WHEN NECESSARY. "In all structures exposed to
g moments of the ordinary materials of construction, such
ons, floors and connections, are not sufficient to resist the
n due to wind pressure, taken in any direction on any part
ditional bracing shall be introduced sufficient to make up
moments.
ES MAY BE INCREASED. "In calculations for wind bracing,
set forth in this code may be increased by fifty per centum.
ESSURE MAY BE DISREGARDED. "In buildings under one
provided the height does not exceed four times the average
wind pressure may be disregarded."

Chicago (1915)

d structures shall be designed to resist a horizontal wind
pounds per square foot for every square foot of exposed sur-
hall the overturning moment due to wind pressure exceed
t of the amount of stability of the structure due to the dead

duced by wind forces combined with those from live and
stress may be increased fifty per cent over those given above;
not be less than required if wind forces be neglected."

Form in general not edited or changed. Some paragraph-captions
litor.

bending due to side pressure; or brackets may be
tioned for the side pressure; or diagonal bracing may be pla
proportioned to transfer the shear of the side pressure to

Base of Column must be Anchored. Where build
tall, so that the overturning due to wind is more than the
unloaded building, the base of column must be anchore
foundation to counteract this upward strain." *

Baltimore (1914)

Wind Pressure. "All new buildings exposed to wind
enough to resist a horizontal wind pressure in any directio
square foot of exposed surface, measuring the entire heigh

Calculation of. "The additional loads caused by th
beams, girders, walls and columns must be determined by
to other loads for such members, as provided for in Secti

Special Bracing. "Special bracing shall be employe
to resist the distorting effect of the wind pressure.

Overturning Moment. "In no case shall the overtu
the wind pressure exceed fifty per cent of the moment
structure."

Magnitude of Unit Stresses Used for Wind-Pres
extracts indicate, it is generally considered proper to us
when allowing for wind-pressure. The practice is based
the highest unit wind-pressure will occur very infr
duration usually will be limited to a very few moments.
that the combined stresses due to wind-loads and dead
not exceed ordinary stresses by more than 50%. If str
wind alone do not exceed 50% of those due to dead and
neglected.

2. Conditions Determining or Affecting W

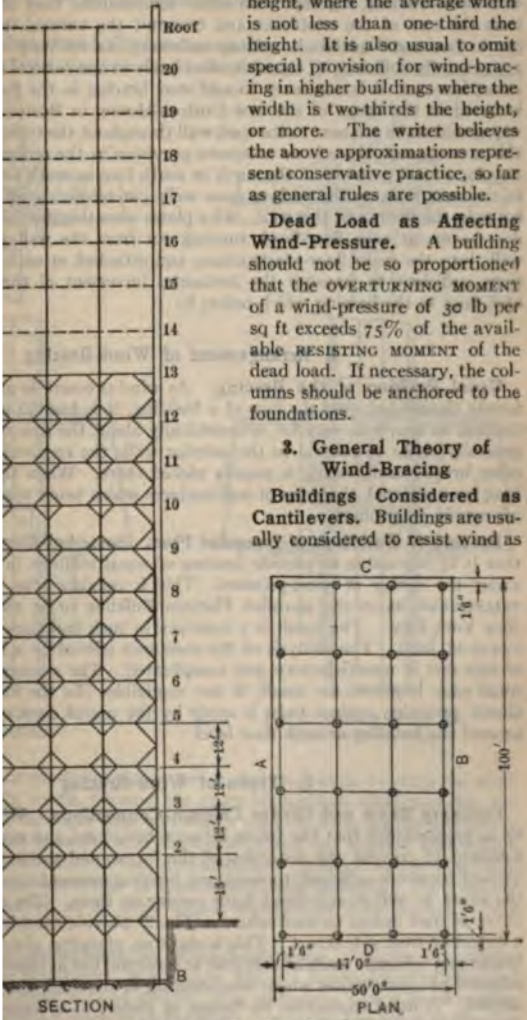
Construction which Resists Wind-Pressure. The
ing, the exterior walls, the interior partitions and the o
beams to columns, all aid in resisting wind-pressure, but

idth as Affecting Wind-Pressure. It is generally safe to ure in structural designs for buildings ten stories or less in height, where the average width is not less than one-third the height. It is also usual to omit special provision for wind-bracing in higher buildings where the width is two-thirds the height, or more. The writer believes the above approximations represent conservative practice, so far as general rules are possible.

Dead Load as Affecting Wind-Pressure. A building should not be so proportioned that the OVERTURNING MOMENT of a wind-pressure of 30 lb per sq ft exceeds 75% of the available RESISTING MOMENT of the dead load. If necessary, the columns should be anchored to the foundations.

3. General Theory of Wind-Bracing

Buildings Considered as Cantilevers. Buildings are usually considered to resist wind as



SECTION PLAN

ig. 1. Section and Plan of Wind-braced Building

ERS or trusses, planted in the earth. Assuming a building nensions shown in section and plan in Fig. 1, with a wind-tion for a wind-pressure on the north or south face to reac in the west face, as the various open wells cut off nearly between the floors and this wall. Flat plates were therefo floor-beams at each floor-level, running out from the wa wells into the main floor-construction, and attached at e tions sufficient to transmit the horizontal increment of each floor to the bracing which resists it.

4. Arrangement of Wind-Bracin

Usual Position of the Bracing. As wind-pressure i formly distributed over the face of a building, it is best t bracing, as nearly as may be, symmetrically about the ax generally easier to conceal in the exterior walls the requi other braces, and bracing is usually placed there. Whe have been selected, the areas of wall-surfaces which bring are readily determined.

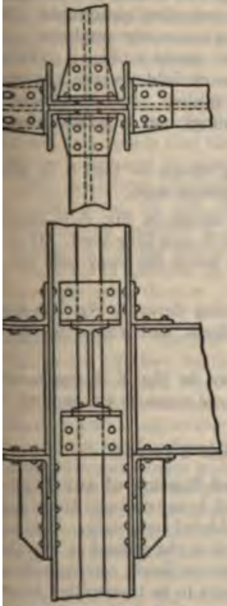
Bracing of Buildings of Irregular Plan. Some build that it is impossible to provide bracing of equal stiffnes about the center of wind-pressure. This is notably tr TRIANGULAR, as in the so-called Flatiron Building or in New York City. The result is a tendency in such buildi VERTICAL AXIS. The analysis of the resistance offered by of this sort is unsatisfactory and complicated. The str usual case, however, are small, if not negligible. In th above, provision against twist is made by the use of dee around the building at each floor-level.

5. Types of Wind-Bracing

Ordinary Beam and Girder Column-Connections. be so proportioned that the joints between horizontal and sufficient to prevent the distortion of the frame, and the vertical members sufficient to resist any bending moments

RESISTING VALUE of such a connection against a moment due to side In lower buildings it is usual to specify two rivets instead of four in ge, and in the case of very high or narrow buildings, six rivets are some- ed.

tance of Beam and Girder-Connections to Wind-Pressure. It is es assumed that the connections of all beams or girders (running in the



. Ordinary Girder and olumn-connections

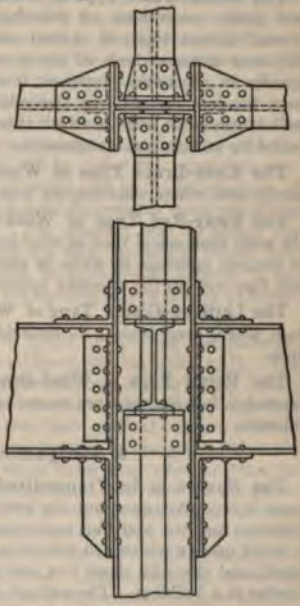


Fig. 3. Heavy Girder and Column-connection

ection as the wind) to columns act at their full value to resist the wind. undoubtedly wrong, because the many connections could probably not to work at the same time, and also because building-frames are seldom so that such a result could be possible, under any rational assumption, to the distribution of the vertical shears. SIDE CLIPS are sometimes the column-connections to furnish additional stiffness. They are not value, however, as on most beams they are not deep enough to help

y Column-Connections in Wind-Resistance. Column-connections times made very heavy, as shown in Fig. 3. A connection of this kind rranged to resist a large TWIST. The RESISTING VALUE is, of course, d by the resisting moment of the rivets connecting the beam to the clip- r by the connection of the angles to the column. This type is used where tance to wind is provided for in a very large number of connections, in all the column-connections, throughout the buildings. Such an arrangement was used in the Hudson Terminal Buildings, New York City. There are several objections to this type of connection. Double beams or girders are required, and the resulting finish is awkward in appearance; the cost, also, of double, compared with single, beams and girders, is high. The additional fireproofing, also, increases the expense, and on the whole, it does not generally prove a satisfactory method of stiffening a building.

The Gusset-Plate Type of Wind-Bracing. In addition to ordinary beam and girder-connections, as described in the preceding paragraphs, there are several distinct types of special wind-bracing commonly employed. Perhaps the most common form is the GUSSET-PLATE, shown in Fig. 4. This is not usually an economical type, as it requires much field-riveting and results in large bending moments in the columns and girders. It accommodates itself well, however, to walls in which there are openings, and is generally easily concealed by architectural treatments.

The Knee-Brace Type of Wind-Bracing shown in Fig. 5 is also commonly used where wind-bracing is placed in exterior walls.

The Sway-Rod Type of Wind-Bracing shown in Fig. 6 is theoretically the most economical type of wind-bracing, but is now little used. It is difficult to arrange openings in walls or partitions in which the sway-rods are placed, and they cut up the masonry considerably.

The Latticed-Girder Type of Wind-Bracing shown in Fig. 7 is sometimes used where deep bracing is desirable for stiffness, and where the stresses are light.

The Portal Type of Wind-Bracing shown in Fig. 8 is cumbersome and expensive, but is sometimes necessary where large openings are required between columns.

6. Computation of Wind-Stresses

The Shears to be Transmitted by Wind-Bracing of any Type are the same in any given case, but the bracing of each type transmits these shears in a different manner, and thus each must be considered separately. It is as though a PLATE GIRDER with SOLID WEB were set on end in the ground, a side thrust exerted, and the WEB THEN CUT AWAY at successive levels corresponding to the stories in a building. The amount of the shears to be transmitted between the flanges would not vary as holes in the web were made, but the road by which the shears traveled would need to be determined by the character of the resulting construction after the holes were cut; and the exact character of the secondary stresses, also, set up in the remaining portions of the web, would depend entirely upon the number and size of holes and their position in the web. The investigation of the shears and moments taken care of by the individual members of a bracing-system may be likened to a study of the secondary stresses in the mutilated web in the imaginary plate girder described above. For a building it is generally convenient to determine the VERTICAL-SHEAR INCREMENTS at each level of bracing, and use these increments in the further analysis of the bending moments and shears in the individual members of the system.

7. Illustration of Method of Computing Wind-Stresses

Thrusts, Vertical Shears and Moment-Increments. If bracing is placed in the walls *C* and *D*, Fig. 1, it is assumed that one-half the length of the building contributes pressure to each line. Let it be further assumed, for the present, that these lines of bracing are the only features of the construction offering resistance to wind-bracing against side *A*. Then, assuming the wind to blow

per sq ft, perpendicular to side A, there are HORIZONTAL THRUSTS
each line of bracing, of 50 by 12 by 30 lb, or 18 000 lb. Re-
L, page 1178, there are found listed in the second column of the
ontal thrusts at each floor. It is assumed that no additional
ached the building below the fourth floor. In the third column
e horizontal thrusts, ΣH, are summarized from the top down to
iving the TOTAL HORIZONTAL THRUSTS. For example, 202 500 lb
ontal thrust down to and including the tenth floor. Each tier
transmit a VERTICAL SHEAR equal to the difference in flange-
point midway in the story above the tier in question, and a
the story below. This difference of flange-stress can, of course,
ertaining the difference in bending moments between the two
ling by the effective depth of the system, as in a plate girder or
w appear that the differences in moments applying to each tier
und and tabulated. These will be called the MOMENT-INCRE-
ave been tabulated for the assumed case in the fifth column of
ourse, the sum of all the moment-increments must equal the
ING MOMENT of the wind. The simplest way to obtain the
nt for any tier is to multiply the total horizontal thrust, ΣH,
el in question by the distance between points midway in the
l below the tier. Thus, for the tenth floor, 202 500 by 12 equals

nts of Vertical Shear are found by dividing the moment-
he effective depth of the cantilever, in this case, 47 ft. The
MENTS are listed in the sixth column of the table. It is
e full depth between outside columns as the effective depth of
This is not strictly correct where there are four or more columns
he bracing, but the assumption is made on the ground that the
urnish flanges which are so many times more effective than the
umns that the latter may be neglected. If there are a number
e plane of the bracing, say six or seven, this assumption becomes
urate, and the effective depth should be reduced. The function
s heretofore stated, is to carry between the flanges at each floor-
ents of vertical shears thus found. The summation of all the
nts from the top down gives the TOTAL VERTICAL LOAD and UP-
d, on the corner-columns, or more correctly, on the outside
rder.

cal Shear. In this assumed case, the total uplift exceeds the
nd live loads on the corner-columns. This, however, is not
there are sufficient means furnished for transferring any excess
into the walls A and B, which act as flanges to the wind-resisting
eral rule, the side walls of a city building are not much reduced
d in higher buildings there are usually spandrel beams in the
r. With such an arrangement a considerable amount of EXCESS
en care of. In some cases special bracing may be necessary, at
anels of walls A and B.

ertical or Flange-Stress. When considering any question
RTICAL LOAD or UPLIFT, such as the one described in the pre-
hs, it should be kept in mind that totals should be used without
y sort. Vertical forces forming couples to resist the wind must
ther they are transmitted through the masonry walls or through
ing. Referring again to the illustration of the plate girder set
the vertical shears are dependent only on the *force of the wind*

and the effective depth of the girder. The exact WEB-STRESS will vary wi form and arrangement of the web, but the TOTAL VERTICAL or FLANGE- must remain the same in any case.

Indeterminate Resistance-Factors. An analysis which makes no al for the resistance of walls, ordinary connections, etc., to wind is fairly and simple, and the bracing can be proportioned with as much precision a structural feature. When the wind-resistance of a building is a primary sideration, as in a tower, the analysis should be made thus, for only in th can a result be obtained, where it is not required to rely on almost unsup judgment for the value of INDETERMINATE FACTORS OF RESISTANCE. W however, ordinary buildings of usual proportions are under consideratio customary and well to make allowance for the INTEDERMINATE FACTORS, best of one's judgment. This is necessary for economy, and is perfectly so long as usual cases are to be dealt with.

Table I. Thrusts, Shears, Moment-Increments, etc., for the Buil Shown in Fig. 1

I	II	III	IV	V	VI	VII
Floor	Horizontal thrust at each floor, H	Total horizontal thrusts from roof to each floor, ΣH	Arm, A	Moment-increment, M	Vertical increment, V	Total vertical increments from roof down, ΣV
	lb	lb	ft	ft-lb	lb	lb
Roof	4 500	4 500	6	27 000	550	550
20	18 000	22 500	12	270 000	5 750	6 300
19	18 000	40 500	12	486 000	10 350	16 650
18	18 000	58 500	12	702 000	14 950	31 600
17	18 000	76 500	12	918 000	19 500	51 100
16	18 000	94 500	12	1 134 000	24 100	75 200
15	18 000	112 500	12	1 350 000	28 700	103 900
14	18 000	130 500	12	1 566 000	33 300	137 200
13	18 000	148 500	12	1 782 000	37 900	175 100
12	18 000	166 500	12	1 998 000	42 600	217 700
11	18 000	184 500	12	2 214 000	47 200	264 900
10	18 000	202 500	12	2 430 000	51 800	316 700
9	18 000	220 500	12	2 646 000	56 400	373 100
8	18 000	238 500	12	2 862 000	61 000	434 100
7	18 000	256 500	12	3 078 000	65 600	499 700
6	18 000	274 500	12	3 294 000	70 200	569 900
5	18 000	292 500	12	3 510 000	74 700	644 600
4	18 000	310 500	12	3 726 000	79 300	723 900
3	....	310 500	12	3 726 000	79 300	803 200
2	....	310 500	15	4 657 500	99 200	902 400
1	....	310 500	15	4 657 500	99 200	1 001 600

Scheme for Developing Special Bracing. The writer offers th as a reasonable and consistent scheme for DEVELOPING SPECIAL BRA such allowances are considered. Unfortunately, it does not seem recommend any method of determining the correct allowances, except eral guides as are mentioned in Subdivision 2, page 1172. In ea some one familiar with construction and usual practice should deci it will be safe to assume the building rigid and secure against ial bracing. In this case, let it be assumed that the building resisting the wind, without the aid of special bracing, as far nth floor. Then, assuming that the walls, beam-connections, ably the same in the floors below, it is fair to say that the rement at the fourteenth floor can be deducted from the at each floor below.

ical Increments. The CORRECTED VERTICAL INCREMENTS er should be used only in the proportioning of special bracing. ig moment of the wind, and the full vertical shears, should ring all other effects of the wind and the resistance of the hould also be borne in mind that this method of proportion- est, largely dependent upon individual opinion, and in any ar better to err on the safe side, even to the extent of disre- the uncertain factors of resistance. The CORRECTED VERTICAL e assumed case have been listed in the eighth column of e flanges of the building, acting as an upright cantilever ssumed concentrated in the outside walls *A* and *B* (Fig. 1), it RTICAL-SHEAR INCREMENTS will be constant from outside to

of Stresses in Different Types of Wind-Bracing

Thrusts, which must be carried by the bracing at each level I and can usually be neglected. The MAXIMUM HORIZONTAL can ori- the ory, ple,

ori- The rac- arry TAL ory, ally ari- ring col- eg-

set- pre- anel ING. e of here

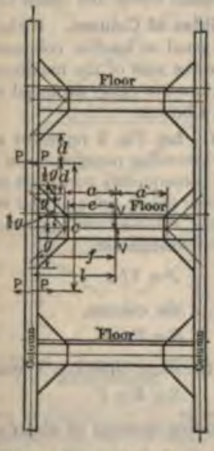


Fig. 4. Gusset-plate Type of Wind-bracing

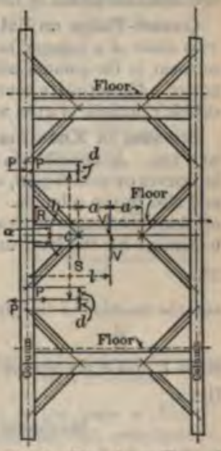


Fig. 5. Knee-brace Type of Wind-bracing

y to distort the frame, changing the angle between the vertical bers; that is, the columns and girders. For this investigation, a s considered, with gusset-plates at either end, as shown. These prevent the distortion, and since they both, at either end, re-

BENDING MOMENT ON THE GIRDER. The flange-stress ha
from this bending moment, it is possible to fix the num
to fasten the flanges to the gusset. The connection of
must provide for a shear equal to V. V multiplied by f
duced through the gusset at the face of the column.
the gusset to the column must be sufficient to resist this m

Points of Inflection occur in the columns midway be
as in the girders. The bending moment in the column ma
mately by assuming the moment exerted through the guss
in the form of a couple acting at points two-thirds of the w
of the gusset to the tips, as indicated. The MAXIMUM BE
COLUMN will then be the horizontal force P multiplied by
multiplying V by l and dividing by c, l being the distan
point in the girder to the axis of the column, and c bein
the inflection-points in the column above and below the

Gusset-Plates on Both Sides of Column. If there
TWO SIDES of a column, as is usual on interior columns, t
moment in the column will be the sum of the maximum
gusset. Gusset-plate connections are easily arranged wit
double channels, or even with **I** beams.

Stresses in Knee-Braces. Let Fig. 5 represent a t
BRACING. As described in the preceding paragraphs on gu
be POINTS OF INFLECTION, and consequently POINTS OF NO
and also in the columns. These points are assumed mic
of the knee-braces in both the columns and girders.
increment for the level under investigation.

Then $$P = Vl/c$$

and the reaction of the girder at the column,

$$R = Va/h$$

the braces at either end of the girder. All web-splices, and
flange-rivets, must be proportioned from these shears.

t of Braces for No Bending Moment in Girder or Column.
om the above that the nearer a and d approach zero the less the
ts in the girder and column become. If the intersections of the
the braces can be arranged so that a and d become zero, there
ING MOMENTS in the girders or columns.

s on One Side, Only, of Girder. It is often necessary to
RACES ON ONE SIDE, only, of the girder, either above or below.
kind the girder itself serves as one arm of the brace, and

$$\text{The stress in the single knee} = S \text{ cosecant } \alpha$$

as determined above, but there must also be taken into account
tress in the girder, due to its action as one arm of the brace.

$$\text{Horizontal stress in girder} = Vl/(\tfrac{1}{2}c - d)$$

on between the column and the girder must provide for the com-
R vertical and $Vl/(\tfrac{1}{2}c - d)$ horizontal.

Sway-Rods. For the correct analysis of SWAY-BRACING (Fig. 6),
ements should be found in a manner slightly different from that
bdivision
-9. The
sures are
, except
pressures
down to
e, in each
nal pres-
as of one-
low. The
ch level
e story-
(See sec-
nd fourth
I.) The
ents are
the other
for these
s; and,
rtical in-
constant
h story.
ANY DI-
the verti-
the story multiplied by the cosecant of the angle α (Fig. 6).
at the diagonals are used for tension only and that, consequently,
acts at a time. Each horizontal member must take compression
tical increment in the story below, multiplied by the cotangent
ints are arranged so that axial lines of members intersect, there
ing either in the columns or the horizontal members.

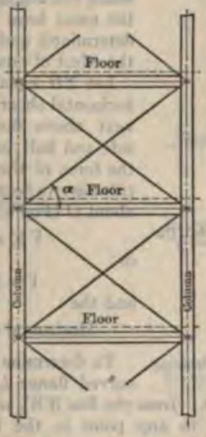


Fig. 6. Sway-rod Type of Wind-bracing

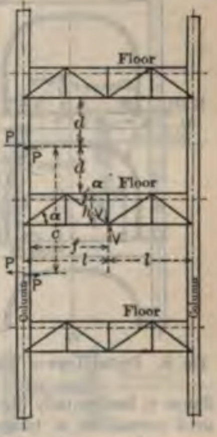


Fig. 7. Latticed-girder Type of Wind-bracing

Latticed Girders. *Let V in Fig. 7 equal the vertical increment*
As in the other types, V is constant between the columns, and
diagonal equals V multiplied by the cosecant of α. As in the

GUSSET-TYPE and KNEE-BRACE TYPE, there is no bending at the of the girder-length, and consequently no stress in the middle top chord. The maximum bending moment in the girder is at th and this

$$\text{Maximum bending moment in the girder} = Vf$$

The maximum chord-stress is at this same point, and this

$$\text{Maximum chord-stress} = Vf/h$$

The connections of the chords to the columns must provide ag imum stress.

$$P = Vl/c$$

and the

$$\text{Maximum bending moment in the column} = Pd$$

Stresses in Portal Bracing. It is not possible to analy stresses in PORTAL BRACING (Fig. 8), when it is used in connectic of continuous section. Th given follows that of C. T. Pu Framed Structures." It is the safe side, and for ordi well be followed. In a large much bracing of this type the exact form of the POR determined, and greater allo the effect of CONTINUOUS CC

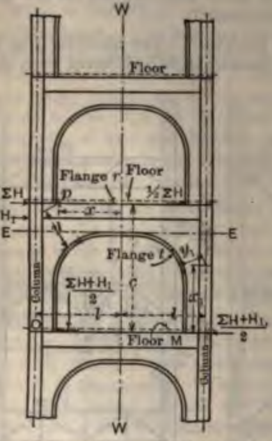


Fig. 8. Portal Type of Wind-bracing

Let ΣH equal the accun horizontal shear from the w next above floor M, appli side and half on the other the force of the wind or th tributary to floor M. Then, about O (Fig. 8)

$$V \times 2\,l = (\Sigma H +$$

or

$$V = (\Sigma H + H_1)$$

and the

Horizontal reaction =

To determine the maxim curved flange t, assume a flange r, horizontally distant x from the line WW, and at the di ured normal to a tangent to any point in the flange t; th center of moments at the left extremity of the distance x, th flange t multiplied by y, equals V multiplied by x, or Vx/y equ the flange t, at the section taken, and this is a maximum when x, est value.

Each leg of the portal, including the column, may be conside LEVER with two forces acting on it, the horizontal force $\frac{1}{2}(\Sigma H$ vertical force $(\Sigma H + H_1)\,c/2\,l$, the flange t (of the right leg for in COMPRESSION and the column itself acting as a TENSION-CHORD point on the axial line of the column, distant x_1 from the bottom at right-angles to, and distant y_1, measured normal to a tangent the flange t, and taking moments about this assumed point, flange t multiplied by y_1 equals $\frac{1}{2}(\Sigma H + H_1)$ multiplied by

uals ½ $(\Sigma H + H_1)$ multiplied by x_1/y_1, and this is a maximum
its greatest value. There is approximation in this treatment,
e side of safety. If the flange t has a section proportioned to
stresses the requirements will be fulfilled. The stress in, and
uired for, the flange r can be obtained in a similar manner. The
he portal above this flange to the portal and column above must
will safely resist the stress ½ ΣH at each leg.

bination of Dead and Live Loads with Wind-Loads

inciples. It usually happens that the same girders that are used
g serve also to carry floors or walls. The dead and live loads
idered with the wind, and the RESULTANT COMBINED STRESSES

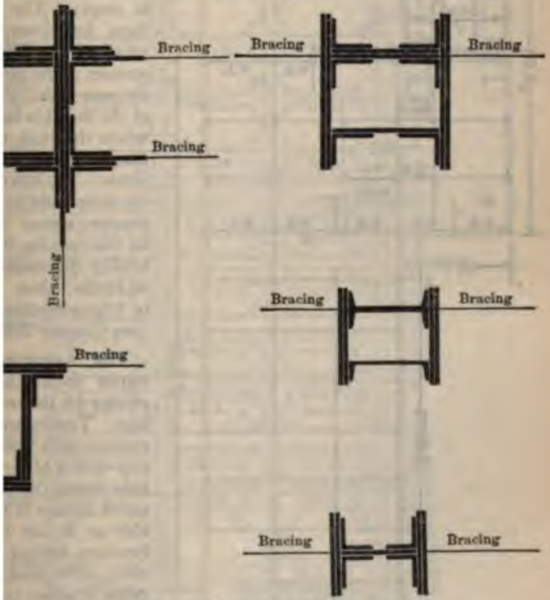


Fig. 9. Types of Columns Arranged for Wind-bracing.

t should be borne in mind that the maximum bending moment
wind is often at a point on the girder more or less removed from
ximum bending moment for dead and live loads. When RESULT-
D MOMENTS are considered, in which the forces are the wind-load
nd dead loads, it is generally deemed proper to use unit stresses
of those of common practice under usual loading. The columns
stigated for direct live, dead and *wind-loads and for the bending*
The RESULTANT STRESS, again, should not exceed 150% of the
y used for live and dead loads only. It is often best to design
pecial view to proper connections for bracing. This aids in

both design and detail. In Fig. 9 are shown a few TYPICAL ARRANG
column-material illustrating this point.

10. Wind-Bracing of Water-Towers and Similar Structur

The Principles Involved in Water-Tower Bracing. In the
TOWER WITHOUT MASONRY WALLS, a problem is presented much sim
that of a building, as the INDETERMINATE FACTORS OF RESISTANCE a
eliminated. The braci
be designed to resis
wind-pressure. It s
borne in mind that
towers the condition of
STABILITY obtains when
is empty. The most
form for tower-braci
SWAY-ROD. The an
stresses is the same as
on page 1181. The ap
of the thrust is largely
where the tank stands
does not in any way alt
alysis. The legs of wat
are frequently SLOPED
greater spread at the
In this case the stresses
readily determined by
METHODS than by
or trigonometrical com
(See Chapter XXVII.

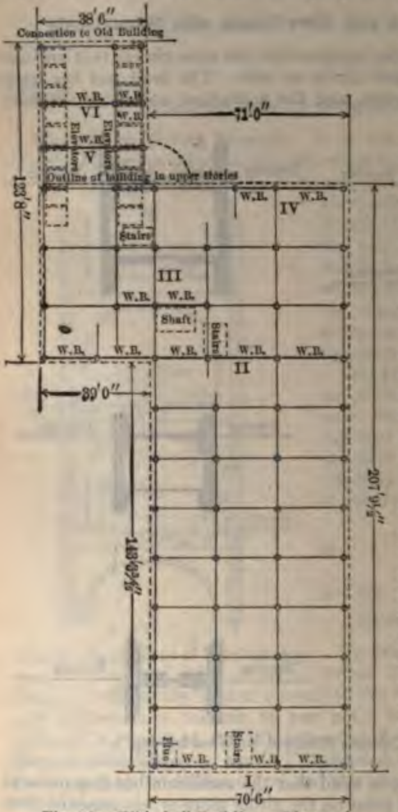


Fig. 10. Whitehall Building. Plan and Lines of Bracing

**The Assumed U
sures** should be so
greater for towers tha
ings. Towers are sma
parison with building
probability of the full
sure being developed
entire surface is grea
ably 40 lb per sq ft
Pressure against a c
BODY, such as a tan
taken at about two-th
full pressure against
tion of the diamet
The stresses under th
pressure should be
usual bounds for ord
or live loads. The a
each post should exceed, by a safe margin, the full uplift due to th
pressure. The weight of water in the tank should not be considered
the uplift.

A Good Example of a Steel Water-Tower is described and Ill
the Engineering Record of June 20, 1903, the stress-diagrams and
struction being given.

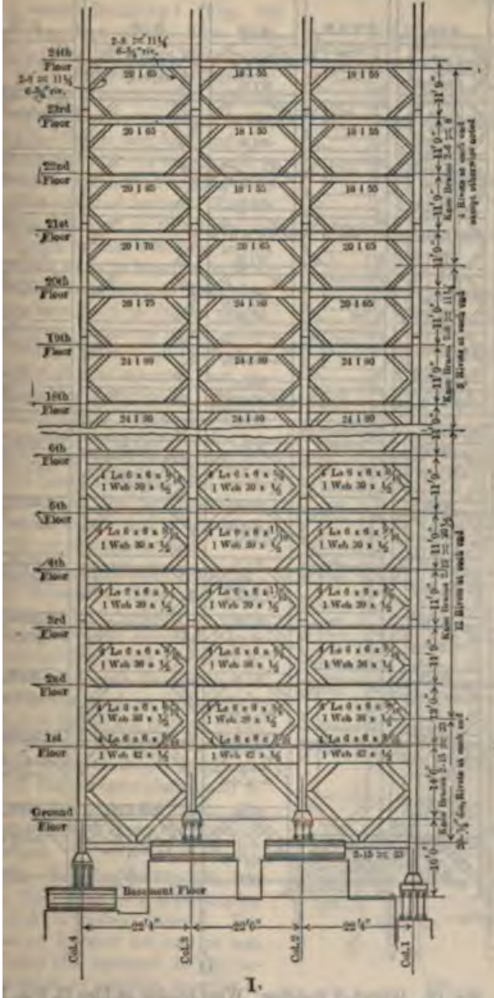


Fig. 11. Whitehall Building. Wind-bracing on Line I, Fig. 10

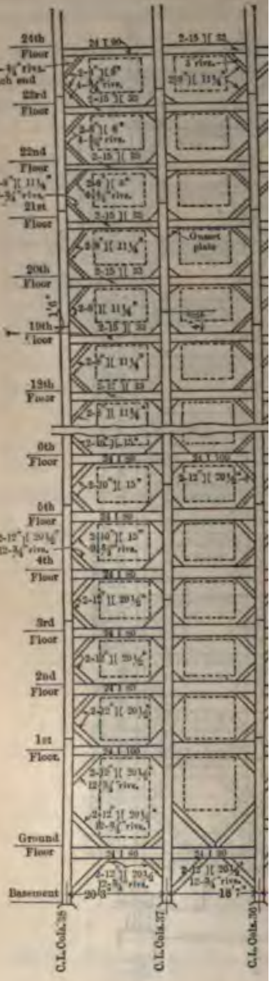


Fig. 12. Whitehall Buildi

it Examples of Wind-Bracing in Tall Buildings*

ll Building† (Figs. 10
West Street, New York
a thirty-one story addi-
r Battery Place Building.
er twenty-story building
uth. As the plan, Fig. 10,
ding is very long and nar-
red with its height and it
n which wind-bracing must
essential feature. The six
ıdicated on the plan (Fig.
n numerals and the letters
so as to interfere as little
e requirements of the plan.
re used as far as practi-
ral instances it was neces-
TS because of the limited
It was assumed that the
ons of girders to columns,
rnished sufficient stiffness
ty-fourth floor-level. Be-
bracing was proportioned
ages 1179–80; that is, al-
e for the INDETERMINATE
TANCE equal to the wind-
venty-fourth floor.

tates Realty Building‡
oadway, New York City,
le of a building in which
is quite essential. It is
s high, and its width is
ared with its length and
was used, as indicated, in
it was not feasible to put
lines to do all the work.
were therefore added be-
he elevator-shafts and in
hown on the plan. No
s used above the fifteenth

Building§ (Fig. 19), 681
w York City, is but twelve
is rather narrow. The
interior lot, and it was
the openings in the ex-
ge as possible, in order to
interior. This, of course,
walls of but little value

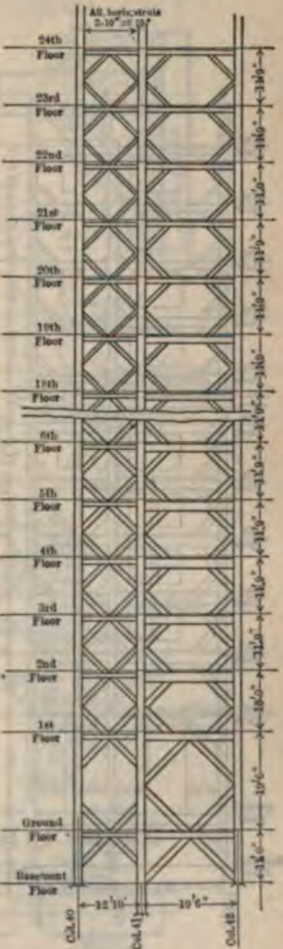


Fig. 13. Whitehall Building. Wind-bracing on Line III, Fig. 10

Henderson acted as designing engineers for these buildings.
& Russell, architects.
H. Kimball, architect.
Mead & *White, architects.*

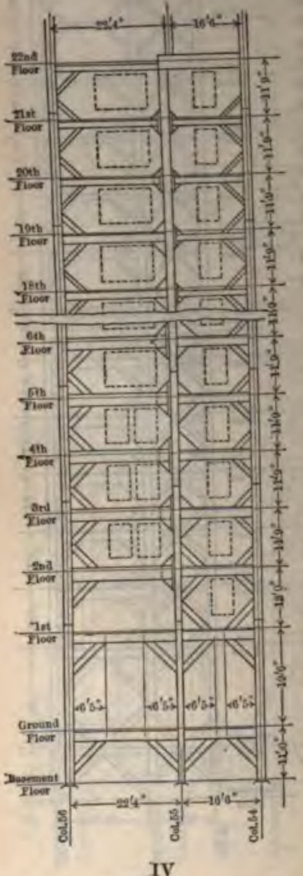


Fig. 14. Whitehall Building. Wind-bracing on Line IV, Fig. 10

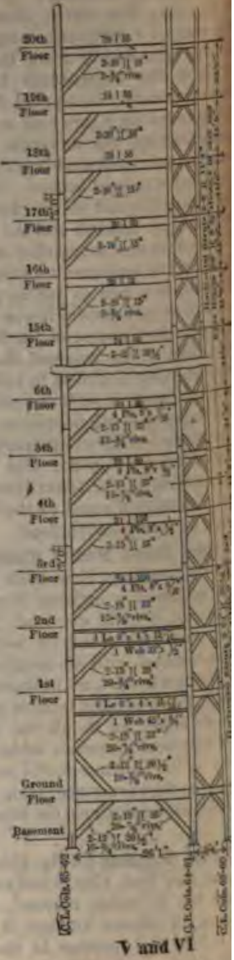
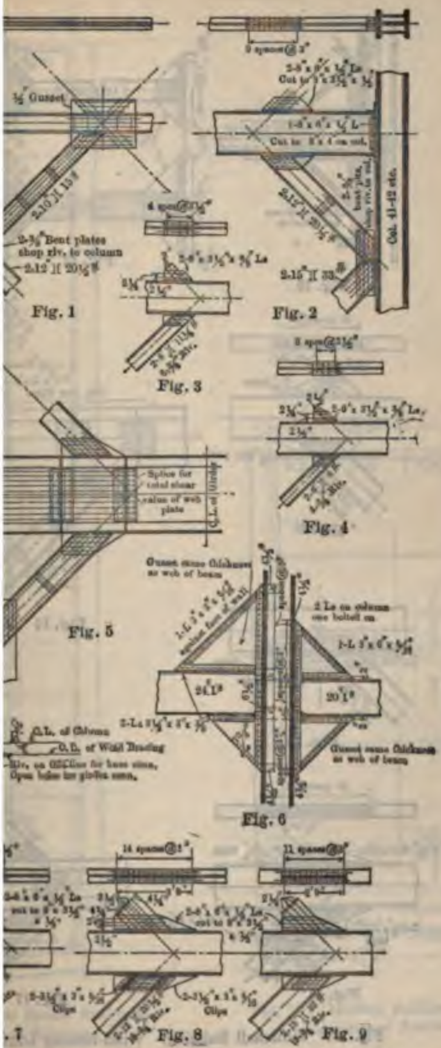


Fig. 15. Whitehall Building
bracing on Line V and VI, F



16. Whitehall Building. *Wind-bracing Details*

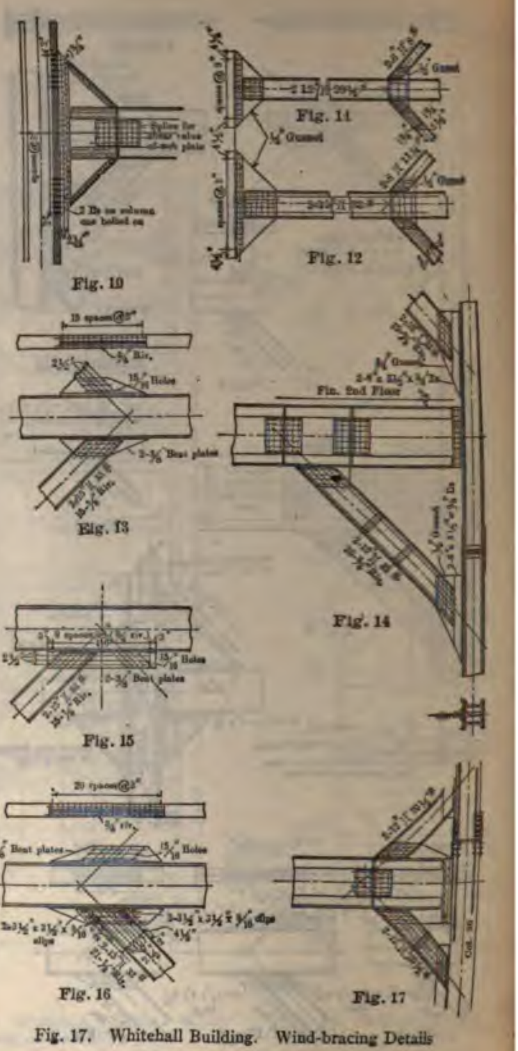
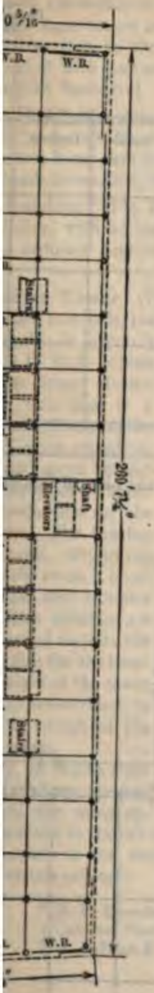


Fig. 17. Whitehall Building. Wind-bracing Details



ates Realty Building.
es of Bracing

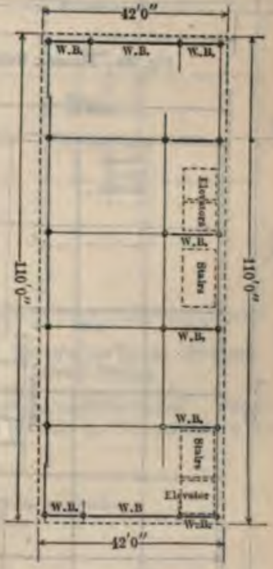


Fig. 19. Morton Building. Plan and Lines of Bracing

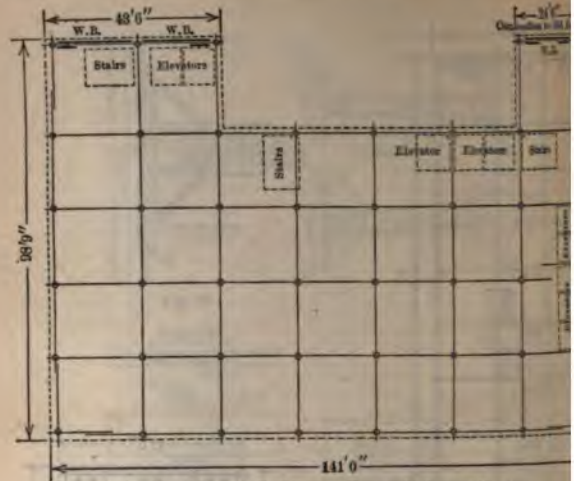


Fig. 20. Masonic Building. Plan and Lines of Bracing

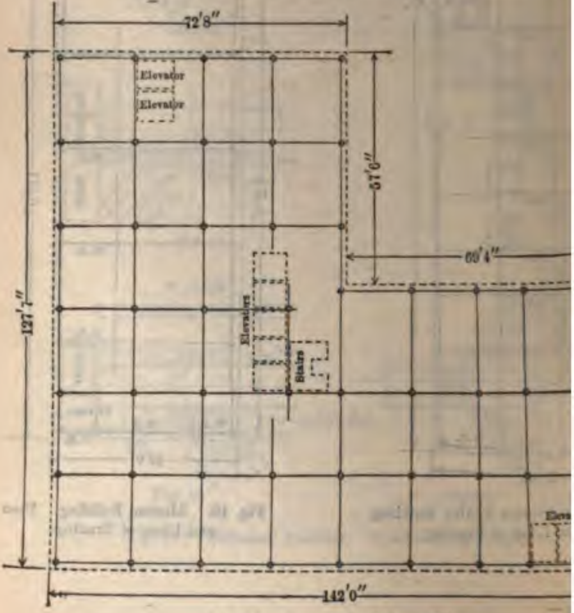


Fig. 21. Everett Building. Plan

KNEE-BRACES and GUSSETS were introduced in each end-
nee-braces next to the elevator-wells and stair-wells. The
ctions to the columns, also, were made with six rivets in
f with four as is usual. These special bracing-features were
h floor-level.

ll* (Fig. 20), 24th Street Building, New York City, twenty-
s virtually no special bracing. Light KNEE-BRACES were
wings, but these were rather to insure the steelwork against
in erection, than to assist in wind-resistance.

ding† (Fig. 21), 45 East 17th Street, New York City, is a
, with no special bracing of any kind. It is large on the
linary features of construction offer ample resistance to

n Tower‡ (Fig. 22), Madison Square, New York City, is
that it is, perhaps, out of place to mention it as an example
It is 700 ft
85 ft in plan
lower stories.
is case is a
structural de-
ch attention.
op to bottom
e of 30 lb per
ction for the
The bracing
, of PLATE
at each level,
and GUSSETS
columns are
l view to the
for the brac-
t of the tower
ESISTANCE to
xcess of the
D.
l drawn from
lings, are per-
he most ap-
ice in respect to wind-bracing as could be chosen. There is
ty in the shape and size of buildings that no case is ever
us example.

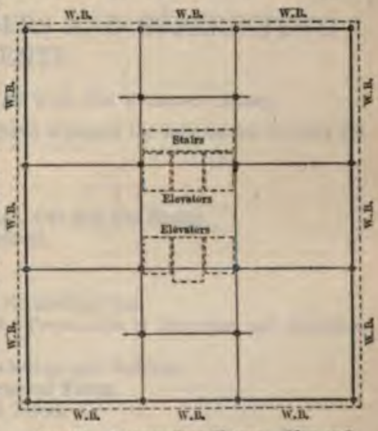


Fig. 22. Metropolitan Tower. Plan and Lines of Bracing

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* H. P. Knowles, architect.
† Goldwin Starrett & Van Vleck, architects.
‡ N. LeBrun & Sons, architects

PART III*

USEFUL INFORMATION

FOR

'ECTS, BUILDERS AND SUPERINTENDENTS

ALL WHO HAVE TO DO WITH THE BUILDING TRADES

author and editors have arranged the information in Part III
g order:
Ventilation.

Plumbing and Drainage, Gas and Gas-Piping.
l Illumination of Buildings.
'k for Buildings.
l Acoustics.
antities and Data for Estimating Cost.
and Data Useful in the Preparation of Drawings and Specifica-

is Information for Architects and Builders.
Architectural and Technical Terms.
tions of Architectural Terms.

t Part III, the subject-matter has been revised, the general order of
s arranged by Mr. Kidder in preceding editions or as submitted in this
sociate editors who have revised divisions of this part of the Pocket-Book.

HEATING AND VENTILATION
AT, FUEL, WATER, STEAM AND AIR

By
ROLLA C. CARPENTER
F EXPERIMENTAL ENGINEERING IN CHARGE OF THE DEPART-
T OF ENGINEERING RESEARCH, CORNELL UNIVERSITY

English unit of heat, or BRITISH THERMAL UNIT (Btu), is the
t required to raise the temperature of 1 lb of pure water 1° F.
curacy, the point on the temperature-scale at which the degree-
ould be specified, and is usually taken either at 39.1° F. or at
ench unit, or CALORIE, is the quantity of heat required to raise
e of 1 kilogram of water 1° C (from 15° to 16° C). Heat and
gy are interconvertible, the MECHANICAL EQUIVALENT OF 1 Btu
y 778 ft-lb. The effect of heat may be noted by: (1) A change
, as indicated by the ordinary thermometer; (2) a change of
ch mechanical work is done; (3) a change of state, such as solid
id to gaseous.

ers. The relative values of the degrees on the different thermom-
given by the following table:

Thermometric Scales

determined	Fahrenheit	Centigrade	Réaumur
en freezing and boiling	180	100	80
t freezing-point........	32	0	0
t boiling-point........	212	100	80
ength of degree........	1	$\frac{9}{5}$	$\frac{9}{4}$
ength of degree........	$\frac{5}{9}$	1	$\frac{5}{4}$
re used................	England and America	France and Germany	Russia

$\frac{9}{5}\text{C} + 32° = \frac{9}{4}\text{R} + 32°$ $\quad\quad$ $\text{C} = \frac{5}{9}(\text{F} - 32°) = \frac{5}{4}\text{R}$

l rule, thermometers are graduated to read correctly for total
the stem emerges into space either hotter or colder than that
lb is placed, a STEM-CORRECTION must be applied to the observed
addition to any correction that may be found in comparison
d.

$$\text{Stem-correction} = 0.000085\, n\,(T - t)$$

observed temperature, t the mean temperature of the emergent
number of degrees of mercury column emergent, and 0.000085
between the coefficient of expansion of the mercury and that of
stem. It has been found by experiment that air at 32° F. con-
part of its volume if its pressure remains constant and its tem-
reased one degree. If this law holds consistently the volume
to nothing at (491.64 − 32) = 459.64° below zero. This point
SOLUTE ZERO, and temperatures measured from it are known
EMPERATURES.

is found in Rhode Island.

Classification of Coals. The usual classification is sh table:

Classification of Coals

Kind of coal	Per cent of combustible	
	Fixed carbon	Volatile matt
Anthracite	97.0 to 92.5	3.0 to 7.5
Semianthracite	92.5 to 87.5	7.5 to 12.5
Semibituminous	87.5 to 75.0	12.5 to 25.0
Bituminous, Eastern	75.0 to 60.0	25.0 to 40.0
Bituminous, Western	65.0 to 50.0	35.0 to 50.0
Lignite*	Under 50	Over 50

* Lignite is an unsatisfactory fuel for furn

Prepared Fuels. Under this head may be mentio pressed fuels, coke and charcoal. The most important o a porous product consisting almost entirely of carbon re manufacturing processes have distilled off the hydrocarb used. It is produced: (1) From gas-coal distilled in gas- or ordinary bituminous coals burned in special furnaces ca (3) from petroleum by carrying the distillation of the re Coke is a smokeless fuel. It readily absorbs moisture fron if not kept under cover its moisture-content may be as mu weight. The heating value of coke per pound of carbon (14 500 Btu. Gas-house coke is generally softer and mor coke, ignites more readily, and requires less draft for its

Determination of Heating Value. The heating valu determined either by a calculation from a chemical anal owing equations express Marks' curves:

$$H = V\left(\frac{7.35}{V - 10} - 0.013\right)$$

HYDROGEN not in moisture for all American coals, to an accu- 0.2 of 1%.

CARBON (carbon occurring in the volatile matter), with an %, approximately:

or $C = 0.9\,(V - 10)$ for anthracite and semianthracite,
$- 14)$ for bituminous and semibituminous coals,
$- 18)$ for lignites.

in coal directly increases the value of V; hence, the calculated will be too high practically by the S content of the combustible. (nitrogen comes off in the volatile matter), with an accuracy :

for anthracite and semianthracite,
$0.012\,V$ for bituminous and lignite.

e of oxygen and sulphur is apparently more or less accidental owing no uniformity, and is not expressible by equations. The ll the sulphur and some of the oxygen will appear in the prox- s volatile, and will, therefore, be accounted for as hydrogen and e of these equations.

FORMULA for calculating the heating value from the chemical is:

$$\text{r pound} = 14\,540\,C + \begin{bmatrix} 52\,500 \\ \text{or} \\ 61\,950 \end{bmatrix} \times \left[H - \frac{O}{8}\right] + 4\,020\,S$$

ts are the heating powers of the separate chemical elements, rogen, giving either the lower or the higher heating value accord- cient used. The formula is in a sense rational, but owing to cies the results obtained are usually from 5 to 10% high. This applicable to gas when another term is added to provide for oxide present. But a simpler method of computation is to take constituents of the gas separately with their calorific values, results. In doing this the following table will be useful:

Calorific Value of Various Gases at 32 Degrees Fahrenheit tmospheric Pressure with Theoretical Amount of Air Required for Combustion

	Symbol	Cubic feet of gas per pound	Btu		Cubic feet of air required per cubic foot of gas
			Per pound	Per cubic foot	
......	H	178.0	62 000	348	2.408
de....	CO	12.81	4 380	342	2.388
......	CH_4	22.4	23 842	1 065	9.57
......	C_2H_6	12.0	22 400	1 865	16.74
......	C_2H_4	12.8	21 430	1 675	14.33
......	C_2H_2	13.79	21 430	1 555	11.93

Approximate Composition and Calorific Value of Some of the Eastern American Coals

State	County	Field, bed or vein	Mine	Moisture	Air-drying loss	Proximate analysis, dry-coal basis			
						Vol.	Fixed C	Ash	Btu
		ANTHRACITES							
Pa.	Lackawanna	....	Scranton	5.41	3.4	7.42	75.90	16.68	12 737
Pa.	Schuylkill	Diamond	Phœnix Park	2.76	1.6	2.55	84.40	13.05	12 933
Pa.	Schuylkill	Mammoth	St. Nicholas	2.80	1.5	1.19	90.75	8.06	13 682
Pa.	Schuylkill	Lykens	West Brookside	3.33	2.6	3.38	87.19	9.43	13 810
R. I.	Newport	500-foot level	Portsmouth	16.80	14.0	2.76	77.44	19.80	11 093
R. I.	Providence	....	Cranston	2.41	2.0	5.04	75.43	19.53	11 268
		SEMIANTHRACITES							
Ark.	Johnson	Spadra	Needmore	2.15	1.4	11.05	78.56	10.39	.....
Ark.	Pope	Shinn Basin	Southern	2.07	1.4	10.02	80.48	9.50	13 991
Pa.	Sullivan	"B"	Connell	3.38	2.6	8.77	79.33	11.90	13 617
Pa.	Sullivan	....	O'Boyle and Fay	3.66	3.0	9.51	76.90	13.59	13 324
Va.	Montgomery	....	Big Vein	2.05	1.6	6.72	59.44	33.84	9 891
Va.	Montgomery	Big Seam	Poverty	4.80	4.1	10.63	70.43	18.94	12 564
		SEMIBITUMINOUS							
Ark.	Franklin	Denning	No. 2	0.84	......	16.60	75.96	7.44	14 769
Ark.	Sebastian	Hartshorne	No. 26	1.99	1.7	16.22	76.58	7.20	14 373
Geor.	Chattooga	Little River	Lookout	2.85	2.3	17.64	74.29	8.07	14 614
Md.	Allegany	Big Vein or Pittsburg	Ocean No. 7	3.42	2.7	18.27	74.39	7.34	14 665
Md.	Garrett	Washington No. 3	"6-foot"	2.33	1.4	16.49	70.07	13.44	13 572
Okla.	Haskell	Hartshorne	San Bois No. 2	2.37	1.9	19.73	71.23	9.04	14 177
Okla.	LaFlore	....	Panama	5.11	4.5	14.39	77.15	8.46	14 398
Pa.	Cambria	Miller	No. 3	3.49	2.8	16.70	77.38	5.92	15 041
Pa.	Cambria	Upper Kittanning	Sunnyside	5.60	5.1	14.87	75.07	10.06	14 074
Pa.	Cambria	Upper Kittanning	Vinton No. 1	1.93	......	19.83	71.8	8.37	14 416
Pa.	Huntingdon	Barnett	Barnett or "B"	2.09	1.6	18.59	75.03	6.38	14 722
Pa.	Indiana	B or Miller	Lackawanna No. 4	2.57	2.1	18.57	70.83	10.60	14 074
Pa.	Somerset	Upper Kittanning	Jenner No. 2	3.99	3.2	16.32	77.14	6.54	14 799
Pa.	Somerset	Lower Kittanning	Kimmelton	3.09	2.6	17.84	70.47	11.69	13 853
Pa.	Somerset	Lower Freeport	Stauffer	2.18	......	18.18	73.63	8.19	14 498

Ky.	Bell	Straight Creek	Straight Creek No. 2	3.10	1.2	[illegible]	[illegible]	4.33	[illegible]
Mich.	Saginaw	Saginaw	Bernard	11.91	7.9	35.76	56.48	7.76	13 374
Mo.	Adair	Bevier	Rombauer No. 2	16.36	14.15	34.82	41.85	23.33	10 769
Mo.	Lafayette	Lexington	Black Diamond	12.34	7.7	39.20	47.87	12.93	12 546
Ohio	Belmont	No. 8	Empire No. 1	4.14	2.6	41.00	49.21	9.79	13 430
Ohio	Guernsey	No. 7	Forsyth	6.65	2.6	36.36	52.34	11.30	13 046
Okla.	Pittsburg	Lower Hartshorne	No. 8	4.45	2.8	37.83	50.66	11.51	13 194
Pa.	Allegheny	Pittsburg	Bertha	3.67	1.8	35.32	59.01	5.67	14 402
Pa.	Allegheny	Upper Freeport	Creighton	2.53	1.0	34.87	55.92	9.21	13 703
Pa.	Cambria	Upper Freeport	Peerless No. 4	3.30	2.7	25.64	65.25	9.11	14 296
Pa.	Clarion	Brookville	Sligo	2.35	1.1	38.37	50.19	11.44	13 435
Pa.	Fayette	Pittsburg	Leisenring No. 1	5.13	4.2	29.38	61.44	9.18	14 089
Pa.	Greene	Waynesburg	Crabapple	2.79	1.2	37.08	49.74	13.18	......
Pa.	Indiana	....	Lucerne No. 1	3.31	2.7	27.13	64.47	8.40	14 241
Pa.	Jefferson	Lower Freeport	Florence	2.53	1.1	28.54	65.61	5.85	14 670
Pa.	Washington	Pittsburg	Blanche	1.70	......	37.84	56.80	5.36	14 584
Pa.	Washington	Pittsburg	Ellsworth No. 1	1.22	......	36.73	56.93	6.34	14 423
Pa.	Westmoreland	Pittsburg	Keystone No. 1	3.98	3.1	29.30	60.12	10.58	13 864
Tenn.	Anderson	Dean	Windrock	6.39	4.7	34.53	55.29	10.18	13 441
Tenn.	Cumberland	....	....	3.53	2.3	21.51	49.60	28.89	10 640
Tenn.	Cumberland	Lower Sewanee (?)	Yellow Creek No. 1	3.89	2.9	28.73	56.26	15.01	13 021
Tenn.	Fentress	Wilder	Fentress	3.03	[illegible]	36.00	50.75	13.25	12 996
Tenn.	Grundy	Sewanee	"B"	5.68	4.7	26.89	53.44	19.67	12 172
Tenn.	White	Sewanee	Clifty No. 1	3.01	1.9	35.78	53.13	11.09	13 511
Va.	Lee	Darby	Darby	3.35	2.0	38.57	56.90	4.53	14 573
Va.	Tazewell	....	....	......	......	......	......	......	......
Va.	Tazewell	No. 4	Richlands	5.62	5.0	24.44	65.18	10.38	14 053
W. Va.	Fayette	Ansted	Gauley Mountain	4.16	2.6	32.64	59.88	7.48	14 384
W. Va.	Harrison	Pittsburg	Pitcairn	1.95	0.5	40.73	51.25	8.02	14 083
W. Va.	Kanawha	No. 2 Gas	Keystone	2.82	2.1	33.13	58.61	8.26	14 166
W. Va.	Monongalia	Upper Freeport	Richard	2.29	1.3	30.56	58.97	10.47	13 876
W. Va.	Tucker	Lower Kittanning	Coketon	2.21	......	24.82	66.99	8.19	14 330

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* Bulletin 22, Bureau of Mines, 1913.

Calorimetric Determinations. The only accurate and
termine the heating value of a fuel is to do so experimentally
For solid fuels, the BOMB CALORIMETER is the most practical.
on the market include the Mahler, the Hempel, the Atwate
These consist essentially of a tight vessel containing a w
oxygen under pressure. This receptacle is placed within
taining a known weight of water and surrounded by heat
to minimize radiation. The sample is EXPLODED electri
absorbed by the surrounding water is determined by mean
thermometer reading hundredths of a degree. Correction
the heat absorbed by the instrument itself, and for radiati

The CARPENTER CALORIMETER, using the principle of the
with heat, works satisfactorily when properly calibrated.

The PARR CALORIMETER uses the bomb-idea, but instead
bustion by compressed oxygen, a chemical powder (sodiu
with the powdered fuel. This instrument is not as reliable a
calorimeter.

The JUNKER CALORIMETER is almost always used for liqui
It consists in burning the fuel within an inverted water-ja
the heat is entirely absorbed by water flowing at a unifo
jacket. The calorimeter is CONTINUOUS, and by metering
in case of liquid fuel), and measuring the rate of flow and t
the jacket-water, the amount of heat given up by the fuel
For a complete description of these calorimeters and their u
Diederichs' "Experimental Engineering."

Temperature of Fire. Of the different methods dev
high temperatures, some of the most important are as foll

(1) **Mercurial Pyrometers, for Temperatures up to 1 0**
point of mercury at atmospheric pressure is 676° F. If so
nitrogen or carbon dioxide, is forced under pressure into
thermometer-stem, the boiling-point is increased. For
the pressure should be about 300 lb.

(2) **Pressure-Pyrometers, for Temperatures up to 1 000°**
an iron bulb filled with ether or hydrocarbon vapor and conn
pressure-gauge. The increase in pressure of the vapor du
moves the dial of the gauge over a calibrated temperature-

(3) **Expansion Pyrometers, for Temperatures up to 1 500**
metals, such as iron and brass, may be utilized to measu
that do not overheat them. The metals must be rigidly
one end, and the other end free to expand separately and mo
a multiplying gear. Such an instrument must be carefully
whole length of the expansion-piece must be at a uniform

erature by the Color of Incandescent Bodies.* "Pouillet concluded andescent bodies have a definite and fixed color corresponding to rature. This temperature-scale follows, and applies only to bodies by incandescent light and not from actual combustion."

Different Colors of Iron Caused by Heat

de	Degrees Fahrenheit	Color
	410	Pale yellow
	430	Dull yellow
	493	Crimson
	502 680	Violet, purple and dull blue; between 261° and 370° C. it passes to bright blue, to sea-green, and then disappears
	932	Commences to be covered with a light coating of oxide, loses a good deal of its hardness, becomes a good deal more impressible to the hammer, and can be twisted with ease
	977	Becomes nascent red
	1 292	Somber red
	1 472	Nascent cherry
	1 657	Cherry
	1 832	Bright cherry
	2 012	Dull orange
	2 192	Bright orange
	2 372	White
	2 552	Brilliant white, welding heat
	2 732 2 912	Dazzling white

moelectric Pyrometers, for Temperatures up to 2 900° F. When wires rent metals are joined at one end and heated, an electromotive force up between the free ends; then if this THERMO-COUPLE is connected te galvanometer of high resistance or a millivoltmeter, the deflec- needle, after a careful calibration, will indicate the temperature very It is only necessary to heat the ELEMENT at the junction. Platinum ys with iridium and rhodium have given the best results.

Melting-Points of Metals

Machinery's Handbook

	Degrees Fahrenheit	Metals	Degrees Fahrenheit	Metals	Degrees Fahrenheit
n..	1 200	Iridium......	4 100	Platinum..	3 200
...	1 150	Iron, cast.....	2 300	Silver.....	1 740
...	500	Iron, wrought	2 900	Steel......	2 500
...	1 700 to 1 850	Lead.........	620	Tin.......	446
...	1 675	Magnesium...	1 200	Titanium .	3 360
a..	2 740	Manganese...	2 200	Tungsten..	5 400
...	2 700	Mercury......	−39	Vanadium.	3 200
...	1 940	Molybdenum.	4 500	Zinc......	785
...	1 930	Nickel.......	2 600	....	....

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* Carpenter's Heating and Ventilating.

(7) **Melting-Points of Metals** which flow at various temperatures up melting-point of tungsten, 5 400° F. A series of metals having melting differing by 100 or 200°, say, are introduced into the furnace, and the ature can thereby be fixed between the melting-points of two of them method is, therefore, only approximate. A table of melting-points fol

(8) **Radiation-Pyrometers, for Temperatures up to 3 600° F.** The he given out by the hot body fall on a concave mirror and are brought to upon the junction of a thermo-couple. The temperature readings tained from an indicator similar to that used with thermoelectric pyrom

(9) **Optical Pyrometers, for Temperatures up to 12 600° F.,** but ordinari in boiler-practice from 1 600 to 3 600° F. The principle on which the instrument is constructed is that of comparing the quantity of light em from the heated body with a constant source of light, in this case a t osmium lamp. The lamp is placed at one end of an optical tube, while other an eye-piece is provided with a scale. A battery of cells furnishes t rent for the lamp. On looking through the pyrometer, a circle of red l pears, divided into distinct halves of different intensities. Adjustment made so that the two halves appear alike and a reading is then taken fr scale. The temperatures are obtained from a table of temperatures corre ing to scale-readings. For standardizing the osmium lamp, an amyl lamp is provided, with a stand for holding the optical tube.

(10) **Determination of Temperature from Character of Emitted Light.** further means of determining approximately the temperature of a furna following table may be of service. The color at a given temperature is a mately the same for all kinds of combustibles under similar conditions.

Character of Emitted Light and Corresponding Approximate Temperat

Character of emitted light	Temperature, degrees Fahrenheit
Dark red, blood-red, low red	1 050
Dark cherry-red	1 175
Cherry, full red	1 375
Light cherry, bright cherry, light red	1 550
Orange	1 650
Light orange	1 725
Yellow	1 825
Light yellow	1 975
White	2 200

Properties of Water. Pure water is a chemical compound of one of oxygen and two volumes of hydrogen. In its natural state it is nev absolutely pure, as it has a greater range of solvent power than any othe For all practical purposes it is non-compressible, the coefficient of co bility ranging from 0.000040 to 0.000051 per atmosphere at ordina peratures. The weight of water depends upon its temperature, and shown by the following table. Maximum density is reached at 39.2° F.

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* Messrs. White and Taylor, Trans. Am. Soc. M.E., Vol. XXI, 1900.

Weight of Water, Freezing to Boiling-Point

Temperature, Fahrenheit	Density, lb per cu ft	Temperature, Fahrenheit	Density, lb per cu ft
32	62.42	130	61.56
45	62.42	135	61.47
50	62.41	140	61.37
55	62.39	145	61.28
60	62.37	150	61.18
65	62.34	155	61.08
70	62.31	160	60.98
75	62.28	165	60.87
80	62.23	170	60.77
85	62.18	175	60.66
90	62.13	180	60.55
95	62.08	185	60.44
100	62.02	190	60.32
105	61.96	195	60.20
110	61.89	200	60.07
115	61.82	205	59.95
120	61.74	210	59.82
125	61.65	212	59.76

er at saturation-pressure, Marks and Davis give the following, which y closely with figures given by Thurston:

Weight of Water at Saturation-Pressure

Temperature, Fahrenheit	Pressure, lb	Density, lb per cu ft
220	17.19	59.63
230	20.77	59.37
240	24.97	59.11
250	29.82	58.83
260	35.42	58.55
270	41.85	58.26
280	49.18	57.96
290	57.55	57.65
300	67.00	57.33
310	77.67	57.00
320	89.63	56.66
330	103.0	56.30
350	135.0	55.57
400	247.0	53.5

s a greater specific heat or heat-absorbing capacity than almost all n substances. Its specific heat is the basis for measurement of the heat-absorption of all other substances. By definition, the SPECIFIC ATER is the number of Btu required to raise the temperature of 1 lb degree. For all practical purposes this may be considered unity, ere is a slight variation with the temperature. The specific heat of . is 0.463.

Properties of Saturated Steam

Pressure, absolute	Temperature, Fahrenheit	Specific volume, cu ft per lb	Heat of the liquid, Btu	Latent heat of evap., Btu	Total he of st Btu
5	162.28	73.33	130.1	1 000.3	[illegible]
8	182.86	47.27	150.8	988.2	[illegible]
10	193.22	38.38	161.1	982.0	[illegible]
11	197.75	35.10	165.7	979.2	[illegible]
12	201.96	32.36	169.9	976.6	[illegible]
13	205.87	30.03	173.8	974.2	[illegible]
14	209.55	28.02	177.5	971.9	[illegible]
14.7	212.0	26.80	180.0	970.4	[illegible]
15	213.0	26.27	181.0	969.7	[illegible]
16	216.3	24.79	184.4	967.6	[illegible]
17	219.4	23.38	187.5	965.6	[illegible]
18	222.4	22.16	190.5	963.7	[illegible]
19	225.2	21.07	193.4	961.8	[illegible]
20	228.0	20.08	196.1	960.0	[illegible]
25	240.1	16.30	208.4	952.0	[illegible]
30	250.3	13.74	218.8	945.1	[illegible]
35	259.3	11.89	227.9	938.9	[illegible]
40	267.3	10.49	236.1	933.3	[illegible]
45	274.5	9.39	243.4	928.2	[illegible]
50	281.0	8.51	250.1	923.5	[illegible]
55	287.1	7.78	256.3	919.0	[illegible]
60	292.7	7.17	262.1	914.9	[illegible]
65	298.0	6.65	267.5	911.0	[illegible]
70	302.9	6.20	272.6	907.2	[illegible]
75	307.6	5.81	277.4	903.7	[illegible]
80	312.0	5.47	282.0	900.3	[illegible]
85	316.3	5.16	286.3	897.1	[illegible]
90	320.3	4.89	290.5	893.9	[illegible]
95	324.1	4.65	294.5	890.9	[illegible]
100	327.8	4.429	298.3	888.0	[illegible]
110	334.8	4.047	305.5	882.5	[illegible]
120	341.3	3.726	312.3	877.2	[illegible]
130	347.4	3.452	318.6	872.3	[illegible]
140	353.1	3.219	324.6	867.6	[illegible]
150	358.5	3.012	330.2	863.2	[illegible]
160	363.6	2.834	335.6	858.8	[illegible]
170	368.5	2.675	340.7	854.7	[illegible]
180	373.1	2.533	345.6	850.8	[illegible]
190	377.6	2.406	350.4	846.9	[illegible]
200	381.9	2.290	354.9	843.2	[illegible]

Properties of Steam. Water boils or vaporizes at a temperature depe on the pressure upon its surface. The thermal units necessary to heat water from 32° F. to the temperature of ebullition is termed the HEAT OF LIQUID and is approximately 1 Btu per degree. The marked characteristi vaporization are: (1) The very great increase of volume at constant temp ture and pressure; (2) the change of the physical state of the material f liquid to vapor; and (3) the enormous quantity of heat absorbed. The absorbed during the process of vaporization is termed LATENT HEAT. W steam is condensed, the same quantity of heat that was received, from wh ever source, is again given off to any substance within its influence w water, iron pipes, etc., colder than itself; and it is this property, tog

of absorbing and retaining heat, that makes water and its vapor
medium for conveying heat from the furnace to the rooms to

am is the steam existing at the temperature and pressure of its

Steam is steam free from liquid in suspension. If the steam
of water it is said to be WET, and the percentage of dryness is
LITY of the steam.

Steam is steam heated above the temperature normal to its
rated steam. The HEAT OF SUPERHEAT may be found by multi-
e of superheat of the steam by its specific heat, approximately

avis' Steam-Tables and Diagrams are generally accepted stand-
e, page 1206.)

poration equals total heat per pound above feed-water temper-
y 970.4, and expresses the number of pounds of steam that
made FROM AND AT 212° F. with the expenditure of an equal

vaporation equals actual evaporation times factor of evapora-
es as a basis for comparison of tests under different conditions
feed-water temperature. Each degree of difference in temper-
ter makes a difference of 0.00104 in the amount of evaporation.

ir is a mixture of oxygen and nitrogen in the proportion of
nd 79.1% nitrogen by volume or 23% oxygen and 77% nitro-
Air in nature always contains other constituents in varying
as dust, carbon dioxide, ozone and water-vapor. The CO_2-
open varies from 4 to 6 parts in 10 000 by volume, and the
from a very small amount to as high as 4% by weight. These
ad out nearly uniformly in the atmosphere by the law of diffu-
property that gases have of mixing and diluting so as to prevent
nt specific gravities from stratifying for any considerable time.
s of the utmost importance to air; for if any inert or poisonous
ain separated in the atmosphere, any one breathing it would be
not poisonous, but if present in large proportions a person might
tion. It is regarded as an index of the quality of the air, and
sent indicates the character of ventilation. It should not exceed
rts in 10 000 for good ventilation. There are various devices in
ermination of CO_2 in air, but the most common consists of a
ing a saturated solution of caustic potash (KOH) into which
ount of air is passed. The KOH absorbs the CO_2, and the
directly on a graduated scale. The degree of moisture in the
rtant influence on ventilation. If the air is saturated no evap-
e place from the body, but if the air is very dry, very rapid
l take place. A mean condition is required for ventilating
t air should be from 50 to 70% saturated in order to feel
percentage of saturation is termed RELATIVE HUMIDITY, while the
f moisture in a given space (usually 1 cu ft) is called the ABSO-

Weights of Air, Vapor of Water, and Saturated Mixtures of Air and Vapor at Different Temperatures, under Atmospheric Pressure of 29.921 Inches of Mercury

Temperature, degrees F.	Volume of dry air at different temperatures, the volume at 32° being 1.000	Weight of cubic foot of dry air at the different temperatures, pounds	Elastic force of vapor in inches of mercury (Régnault)	Mixtures of air saturated with vapor						Cubic feet of vapor from one pound of water, at its own pressure in column 4
				Elastic force of the air in the mixture of air and vapor in inches of mercury	Weight of cubic foot of the mixture of air and vapor			Weight of vapor mixed with one pound of air in pounds	Weight of dry air mixed with one pound of vapor in pounds	
					Weight of the air in pounds	Weight of the vapor in pounds	Total weight of mixture in pounds			
1	2	3	4	5	6	7	8	9	10	11
0	0.935	0.0864	0.044	29.877	0.0863	0.000079	0.086379	0.00092	1092.4	....

generally used for humidity-determinations consists merely eters mounted side by side so as to be swung from a handle. is covered with a piece of moistened cloth to keep the atmos- aturated, and the other bulb is exposed to normal conditions. rs will read differently by an amount depending upon the wn by the following table condensed from the Psychrometric partment of Agriculture:

tive Humidity, Per Cent, for Barometer 30 Inches

Depression of wet-bulb thermometer, degrees																	
3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
69	59	49	39	30	20	11	2	...	...	...	...	...	...	...	...	...	...
72	63	54	45	36	27	19	10	2	...	...	...	...	...	...	...	...	...
75	68	60	52	45	37	29	22	15	7	0	...	...	...	...	...	...	...
78	71	64	57	51	44	38	31	25	18	12	6	...	...	...	...	...	...
80	74	67	61	55	49	43	38	32	27	21	16	10	5	0	...	...	...
82	76	70	65	59	54	49	43	38	33	28	23	19	14	9	5	0	...
83	78	73	68	63	58	53	48	43	39	34	30	26	21	17	13	9	5
85	80	75	70	66	61	56	52	48	44	39	35	31	27	24	20	16	12
86	81	77	72	68	64	59	55	51	48	44	40	36	33	29	25	22	19
86	82	78	74	70	66	62	58	54	51	47	44	40	37	34	30	27	24
87	83	79	75	72	68	64	61	57	54	50	47	44	41	38	35	32	29
89	85	81	78	74	71	68	65	61	58	55	52	49	47	44	41	39	36
89	86	83	80	77	73	70	68	65	62	59	56	54	51	49	46	44	41
90	87	84	81	78	75	73	70	67	65	62	60	57	55	52	50	48	46
91	88	85	82	80	77	74	72	69	67	65	62	60	58	55	53	51	49

23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
2	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
9	6	3	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
15	12	9	7	4	1	...	...	...	...	...	...	...	...	...	...	...	...
20	18	15	12	10	7	5	3	0	...	...	...	...	...	...	...	...	...
29	26	24	22	19	17	15	13	11	9	7	5	3	1	...	...	...	...
35	33	30	28	26	24	22	21	19	17	15	13	12	10	8	7	5	4
40	38	36	34	32	30	28	26	25	23	21	20	18	17	15	14	12	11
43	41	40	38	36	34	33	31	29	28	26	25	23	22	21	19	18	17

at of air for ordinary temperatures, as determined by Régnault, ce one Btu will raise the temperature of 4.2 lb of dry air F.) 1° F. As air for ventilation contains more or less moisture, be warmed, 55 cu ft at 70° F., or 50 cu ft at 32° F., is gener- he equivalent of 1 lb of water in heating. Furthermore, since atmospheric pressure condensed to water gives off about 970 n approximately 50 000 cu ft of air 1° F.

Linear Expansion of Solids at Ordinary Temperatures

British Board of Trade, from Clark

Substances	For 1° F.	For 1° C.	Coef. of expansion from 32° to 212° F.
	Length = 1	Length = 1	
Aluminium (cast)	0.00001234	0.00002221	0.002221
Antimony (cryst.)	0.00000627	0.00001129	0.001129
Brass, cast	0.00000957	0.00001722	0.001722
Brass, plate	0.00001052	0.00001894	0.001894
Brick	0.00000306	0.00000550	0.000550
Bronze (Cu, 17; Sn, 2½; Zn, 1)	0.00000986	0.00001774	0.001774
Bismuth	0.00000975	0.00001755	0.001755
Cement, Portland (mixed), pure	0.00000594	0.00001070	0.001070
Concrete: cement, mortar and pebbles	0.00000795	0.00001430	0.001430
Copper	0.00000887	0.00001596	0.001596
Ebonite	0.00004278	0.00007700	0.007700
Glass, English flint	0.00000451	0.00000812	0.000812
Glass, thermometer	0.00000499	0.00000897	0.000897
Glass, hard	0.00000397	0.00000714	0.000714
Granite, gray, dry	0.00000438	0.00000789	0.000789
Granite, red, dry	0.00000498	0.00000897	0.000897
Gold, pure	0.00000786	0.00001415	0.001415
Iridium, pure	0.00000356	0.00000641	0.000641
Iron, wrought	0.00000648	0.00001166	0.001166
Iron, cast	0.00000556	0.00001001	0.001001
Lead	0.00001571	0.00002828	0.002828
Magnesium	....	....	....
Marbles, various from	0.00000308	0.00000554	0.000554
Marbles, various to	0.00000786	0.00001415	0.001415
Masonry, brick from	0.00000256	0.00000460	0.000460
Masonry, brick to	0.00000494	0.00000890	0.000890
Mercury (cubic expansion)	0.00009984	0.00017971	0.017971
Nickel	0.00000695	0.00001251	0.001251
Pewter	0.00001129	0.00002033	0.00203
Plaster, white	0.00000922	0.00001660	0.00166
Platinum	0.00000479	0.00000863	0.00086
Platinum, 85% Iridium, 15%	0.00000453	0.00000815	0.00081
Porcelain	0.00000200	0.00000360	0.00036
Quartz, parallel to major axis, *t* 0° to 40° C.	0.00000434	0.00000781	0.00078
Quartz, perpendicular to major axis, *t* 0° to 40° C.	0.00000788	0.00001419	0.00141
Silver, pure	0.00001079	0.00001943	0.00194
Slate	0.00000577	0.00001038	0.00103
Steel, cast	0.00000636	0.00001144	0.00114
Steel, tempered	0.00000689	0.00001240	0.00124
Stone (sandstone), dry	0.00000652	0.00001174	0.00117
Stone (sandstone), Rauville	0.00000417	0.00000750	0.00075
Tin	0.00001163	0.00002094	0.00209
Wedgewood ware	0.00000489	0.00000881	0.00088
Wood, pine	0.00000276	0.00000496	0.00049
Zinc	0.00001407	0.00002532	0.00253
Zinc, 8 Tin, 1	0.00001496	0.00002692	0.00269

Cubical expansion, or expansion of volume = linear expansion ×

Systems of Heating

s systems employed for the warming of buildings, aside from the and fireplaces, may be classified as follows:

ting........ Natural-draft, or gravity-system
Forced-draft, or fan-system

ng......... Gravity, or low-pressure systems
  (1) Direct radiation
  (2) Direct-indirect radiation
  (3) Indirect radiation
Vacuum-systems
  (1) Paul system
  (2) Webster system
High-pressure systems
  (1) Gravity-circulation with return-trap or pump
  (2) Fan, or forced-blast system

eating...... Open system
  Direct or indirect
Closed system
  Direct or indirect

ms are briefly described in the following pages and sufficient data le an architect to specify or design, in a general way, an ordinary . The limits of the book preclude the going into many of the minor are usually left to the judgment of the contractor, or to a discus- gh-pressure systems, which are generally employed only for very s or for power-plants, and for which the plant should be designed . For further information on this subject the reader is referred d Ventilating Buildings, by R. C. Carpenter.

Gravity Systems of Steam-Heating

eating Plant may be divided into three distinct parts: (1) The am-generator; (2) the radiators; and (3) the supply and return- ing the two.

Radiators are generally made of iron, and may be of any shape of a good circulation of steam through them, and also permit the e freely about the outside. It is also desirable that the thickness hall be only enough to give sufficient strength.

Radiators. Radiators are divided into three classes, those afford- ct radiation; (2) direct-indirect radiation; (3) indirect radiation.

ting Surfaces embrace all heaters placed within a room or hall ir already in the room.

liating Surfaces embrace heating-surfaces placed outside the rooms and should only be used in connection with some system of ven-

ect Radiation is a mean between the other two methods. The placed in the rooms to be heated, as in *the first method,* and a h air brought *to them through openings in the outside* wall of the *gh a space under the lower sash of a window,*

Efficiency of Radiators. The condensation of one pound of steam
pheric pressure to water at 212° F. gives out 970 thermal units.
determine the amount of heat given out by any radiator in a given
only necessary to determine the amount of water in pounds which th
condenses in the same time and multiply it by 970. The radiator wh
the same conditions of steam-pressure, volume and temperature of
air, will condense the most water in a given time is the most efficient.

Measurement of Radiators. Radiators are rated, or measured
cording to their size, but according to the amount of heating-surfac
in contact with the air. The size of radiator for a given amount o
surface will d
tirely upon th
shape of the r

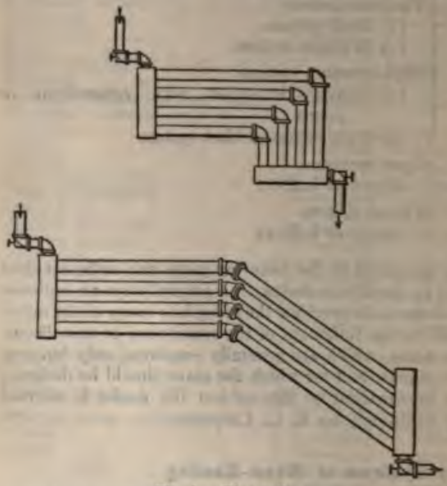


Fig. 1. Pipe-coils

Heating by
Radiation. D
diation being m
economical th
radiation, it w
be much more
used for stea
water heating
buildings not
a great amou
tilation it offe
perfect mode

Description
Coils. The
direct radiat
formed of w
pipes, 1-in p
generally
placed agai
one above th
connected w
bends or bra
elbows, to afford a circulation. The length of pipe required to mak
amount of heating-surface can easily be determined by the use of t
pages 1274–6. For rooms in which it is desirable that the heating-ap
present a neat appearance and occupy as little space as possible som
right radiator is generally employed. Fig. 1 shows pipe-coils made
and manifolds. The piping must turn an angle to allow for the un
sion of the top and bottom pipes. Pipe-radiators are formed of
short upright 1-in tubes from 2 ft 8 in to 2 ft 10 in long, screwed into a
iron base or box, and are either connected in pairs by return-be
upper ends or else each tube stands singly, with its upper end closed
a hoop-iron partition extending up inside it from the bottom almo
The radiators are also made circular in form, either in one piece, or
encircling iron columns. The table following shows the dimens
pipe-radiators for different heating-surfaces.

Cast-Iron Direct Radiators. Until quite recently direct ra
made almost exclusively of cast iron. Since about 1905 con
provement has been made in the design and quality of cast-iron

atterns have very largely superseded those made previous to

manufacturers of radiators are the American Radiator Com-
tler & Pierce, Fowler & Wolfe Manufacturing Company, and
Company, all of whom make several complete lines. There
r of smaller companies who make two or three styles.

Table of Vertical Pipe-Radiators

bes ach w	Surface in sq ft*	Length, ft in	No. of rows and width of base	Tubes in each row	Surface in sq ft*	Length, ft in
4	4	0 10¼	Two rows, width of base, 6¼ in	8	16	1 6¼
6	6	1 2¼		10	20	1 10¼
8	8	1 6¼		12	24	2 2¼
0	10	1 10¼		14	28	2 6¼
2	12	2 2¼		16	32	2 10¼
6	16	2 10¼		18	36	3 2¼
0	20	3 6¼		20	40	3 6¼
4	24	4 2¼		24	48	4 2¼
8	28	4 10¼		28	56	4 10¼
2	32	5 6¼		32	64	5 6¼
8	38	6 6¼		38	76	6 6¼
8	24	1 6¼	Four rows, width of base, 10 in	4	16	0 10¼
2	36	2 2¼		8	32	1 6¼
6	48	2 10¼		12	48	2 2¼
0	60	3 6¼		16	64	2 10¼
4	72	4 2¼		20	80	3 6¼
8	84	4 10¼		24	96	4 2¼
2	96	5 6¼		28	112	4 10¼
8	114	6 6¼		32	128	5 6¼

* For radiators 35 in high.

e in Square Feet per Section of American Direct Radiators

iator	Width at base	Length per section	Height of radiator in inches											
			45	38	32	26	23	22	20	18	16	15	14	13
n……	5⅜	2½	..	3	2½	2	1⅔	..	1½	…	…	…	..	…
ns……	8³⁄₁₆	2½	5	4	3⅓	2⅔	2⅓	..	2	…	…	…	..	…
ns……	9⅞	2½	6	5	4½	3¾	…	3	…	2¼	…	…	..	…
ns……	11⅜	3	10	8	6½	5	…	4	…	3	…	…	..	…
y……	12½	3	..	..	…	…	…	..	5	…	3¾	…	..	3
nn……	5⅝	2½	..	3	2½	2	1⅔	..	1½	…	…	…	..	…
nns……	8¼	2½	5	4	3⅓	2⅔	2⅓	..	2	…	…	1½	..	…
nns……	9¾	2½	6	5	4½	3¾	…	3	…	2¼	…	…	..	…
nns……	11¼	3	10	8	6½	5	…	4	…	3	…	…	..	…
al……	8¼	3	5	4	3⅓	2⅔	2⅓	..	2	…	…	…	..	…
……	8½	3	..	7	5¾	4½	…	..	3¼	…	…	…	..	…
……	8⅝	2½	..	4	3⅓	2⅔	…	..	2	…	…	…	..	…
……	12½	3	..	..	…	…	…	..	6	5⅓	4⅔	…	4	3⅔

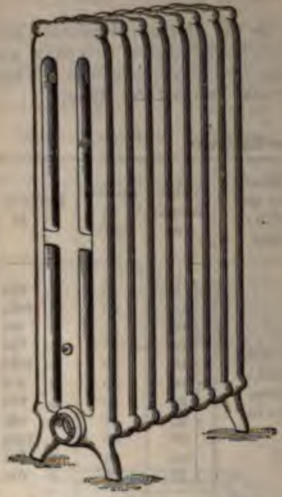


Fig. 2. Rococo Three-column Radiator

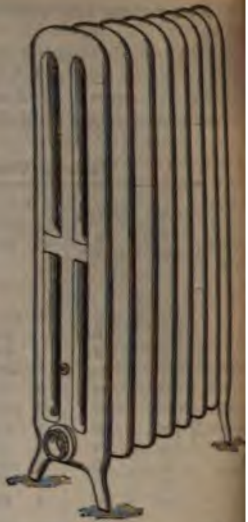


Fig. 3. Peerless Three-column

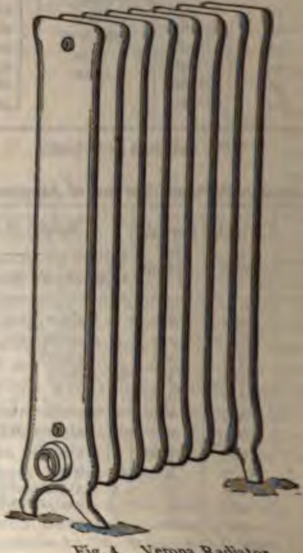


Fig. 4. Verona Radiator

iators made by the American Radiator Company, however, are prob- extensively used than those of any other make, and it is for this t they have been selected for illustration. Nearly all of the patterns his company are very closely duplicated by the companies above named,

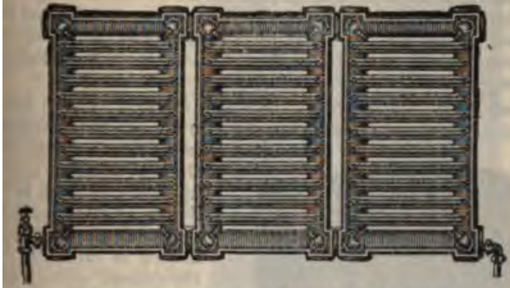


Fig. 5. Three Sections of Colonial Wall-radiator

ion being principally in the ornamentation. Figs. 2, 3 and 4 illus- e of the most popular styles of radiators made by this company, a large variety of radiators in one-column, two-column, three-column olumn and in extended single-column and flue-construction are also

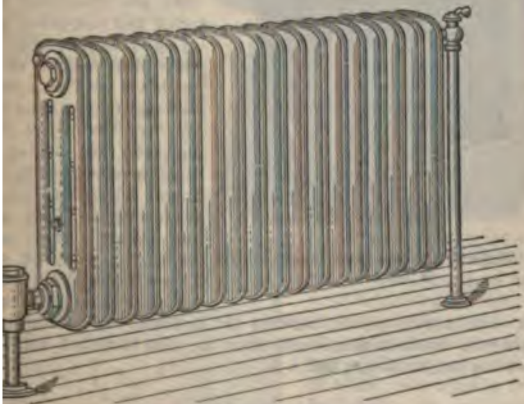


Fig. 6. Presto Pressed Metal Radiator

hem. Fig. 5 shows three sections of the Colonial wall-radiator made npany, which is very convenient *for use in halls and bath-rooms,* as only *from 3¼ to 4½ in from the wall. Corner, circular, curved,* *-radiators; dining-room, window, stairway, box-base,* and direct-

indirect radiators; such auxiliaries as brackets, pedestals, tops, d
wall-boxes; special radiator-sections with high, low, or single legs a
by the American Radiator Company, and by all of the other com
mentioned.

To find the number of sections required, divide the required heati
feet by the values given in table on page 1213. To find the length
ator, multiply the number of sections by the length per section and
two bushings. Radiators are generally put together at the factor
The standard height, except for window-radiators, is 38 in. Heig
38 in cost a little more per square foot.

Pressed Metal Radiators.* The Presto radiators (Fig. 6)
tured and sold under the trade name of Presto Pressed Metal Ra
essential advantages
them are efficiency a
in heating, lightness i
lb per sq ft as aga
cast iron), a uniform
method of rating, b
the decimal system,
teed actual surface-m
rapid heating and q
large air-spaces betw
smooth surfaces ins
side, occupying fro
to one-third less sp
of cast iron. It is
that they are unb
they are guaranteed
proof and rust-proof
of 99.672% pure iro
their light weight
models of the Presto
radiators may be hun
clear of the floors
brackets, thus makin
smooth outside surf
more convenient
arrangement. They
one column, from
high, with a 1½-in
tween the sections;
and four columns, fr
in high, with a 2-in spacing between the sections; and in addi
furnished with brackets mentioned, they are also furnished wit
legs, in three heights of 4, 5 and 6 in.

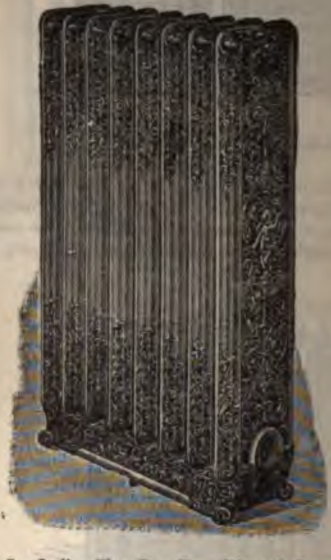


Fig. 7. Italian Flue Box Base Direct-indirect Radiator

Direct-Indirect Radiation

Heating by Direct-indirect Radiation. The only difference
method of heating and the direct method is that external air is int
the room in such a way that it shall come in contact with the r
becoming heated, circulate through the room, and unless other m

---

* Manufactured by the Pressed Metal Radiator Company, Pitts

hrough the cracks around the doors and windows. By this
ficient ventila-
or living-rooms
th direct radia-
tion at all is
e are several
ging the radia-
inlets, although
that the radia-
ted against an
The simplest
iding direct-in-
is by using a
the lower por-
as to form a
Fig. 7. Cold
ucted from the
house and ad-
bottom of the
Fig. 8. It is
pass upward
iator-flues their
nd is brought
comparatively
re. A small
placed in the
, and a damper
ut in the cold-
at the radiator
l into the ordi-
by simply closing the damper and opening the doors. This would probably be required in very cold weather. The outside of the radiator, of course, heats by direct radiation at all times. If a large amount of ventilation is required, some form of indirect radiator should be enclosed in an incombustible casing and the outside air admitted below the radiator. A very good arrangement to accomplish this purpose is shown in Fig. 9. It consists of a stack of pin or other indirect radiators enclosed in a box of either iron, marble, or wood lined with tin, and provided with registers at the top for the escape of the heated air. The cold air enters through a hollow iron sill placed *above the wooden sill of a window and passes down*
ator, through a galvanized-iron pipe, to the space under the

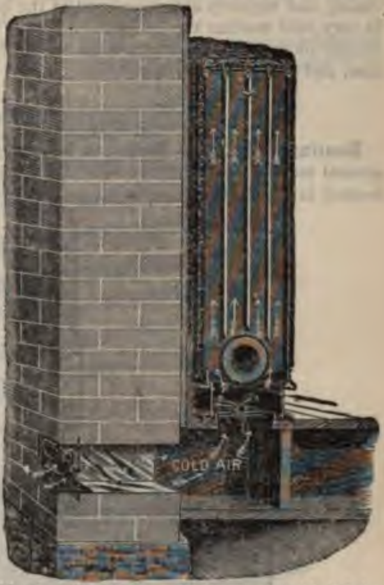
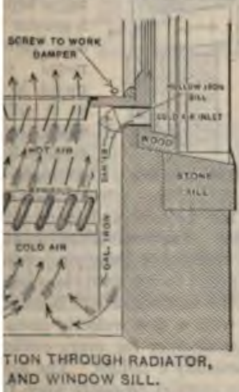


Fig. 8. Direct-indirect Radiator and Cold-air Inlet



ection Through Radiator, Cas-
and Window-sill

radiator. The cold-air inlet is provided with a damper so th
closed, and registers are also placed at the base of the radiator-c
in very cold weather the cold-air inlet may be partially or whol
the air allowed to circulate through the bottom register, up thro
ator, and out of the top registers.

Indirect Radiation

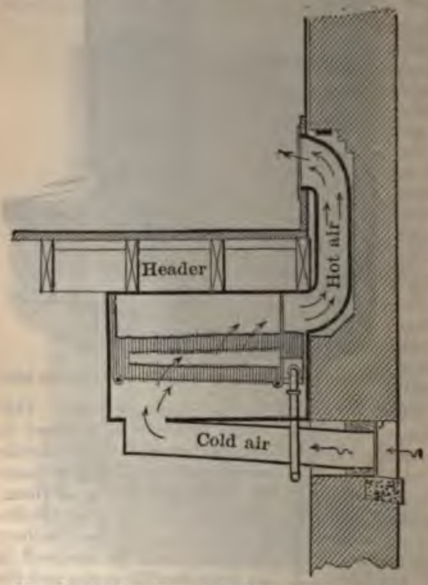


Fig. 10. Section Through Indirect Radiator-stack

Heating by Indirect Radiation is accomplished by two metho
general method being the one which includes separate radiators f
located in the cellar or basement, incased with metal, or wood l
and provided
air inlet and
convey the h
room to be
other method
one cold-air
whole buildin
coil of steam-
behind it, so
air entering
must pass
coil. Such a
only be used
with fan venti
10 shows the
of casing indir
The casing is
galvanized iro
lined with tin.
is better when
to be kept co
a greater loss
and conducti
metal cases;
metal is to
as it will no
when put t
small bolts ca
to make rep
damage.
should be fitted with a door in the bottom, and the cold-air
always be provided with a damper. The vertical air-ducts ar
flues built into the wall when the building is going up. Someti
only plastered; but round, smooth metal linings with close joints g
best results. The cross-section of the air-duct should be compar
as a large volume of warmed air with a slow velocity gives the
There should be a separate vertical air-duct for every outlet or
branched vertical air-ducts one is generally a failure. The heated
heater may be taken to two or more vertical air-ducts when they
over it; the duct to the lower room being taken from the top and
upper room from the side, or both from the top. If both rooms
same level, both ducts should be taken from the top of the box.
air ducts are best when there is one for every coil or heater.

-branched cold-air duct is used, but this system will give trouble the rooms are ventilated by forced ventilation.

diators. For indirect heating a form of radiator is employed different e used for direct radiation. In this method the desideratum is to

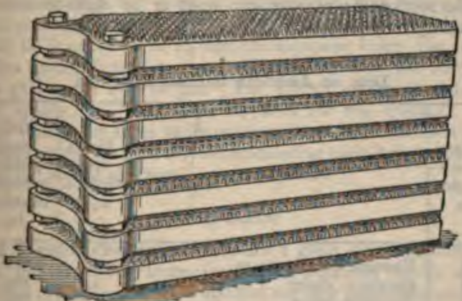


Fig. 11. Perfection Indirect Pin Radiator

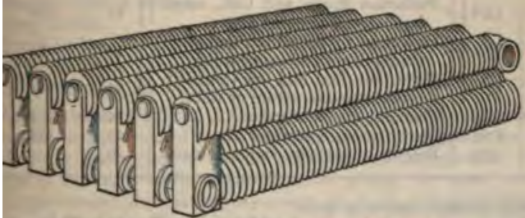


Fig. 12. Excelsior Indirect Radiator for Steam

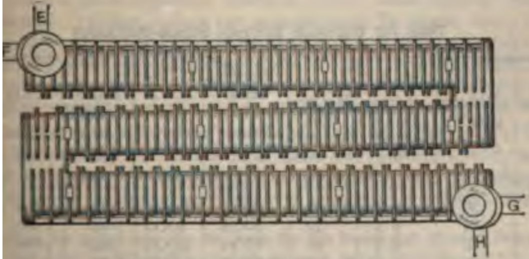


Fig. 13. Sterling Indirect Radiator

any square feet of heating-surface in as little space as possible, appearg of no importance. The earliest form used, and which is still used n or hot-blast systems, is the pipe-coil radiator, formed of a coil of ected at the ends with return-bends. For ordinary indirect heating diators of one of the types shown in Figs. 11, 12 and 13 are now used almost exclusively, as they are fully as cheap if not cheaper than radiators and more satisfactory. The pin-radiator is made by several facturers and is one of the earliest types of indirect radiators. That in Fig. 11 is made in two sizes, STANDARD, with 10 sq ft of heating- and EXTRA LARGE, with 15 sq ft of heating-surface. They are also made connecting the sections by FLANGE AND BOLT or by right-threaded and threaded couplings.

Data on American Indirect Radiators

Length of section, in	Extreme height, in	Pattern-name	Heating-surface, sq ft	Width each section occupies in stack, in
23¾	8	Excelsior junior, steam	8	3⅛
36	8	Excelsior, steam	12	3⅛
36¾	8	Excelsior, water	12	3⅛
36¼	11½	Perfection flange and bolt, standard size, steam or water	10	2¾
36¼	15½	Perfection flange and bolt, extra-large size, steam or water	15	2¾
36¼	9¹³⁄₁₆	Perfection right and left threaded, standard size, steam or water	10	2¾
36¼	14	Perfection right and left threaded, extra large size, steam or water	15	2¾
37¼	11¼	Cardinal, steam or water	15	3½
36¾	15¾	Sterling, steam or water	20	3½
36⅛	15¼	Sanitary school pin, steam or water	20	4

* Bushing reduction cannot be made.

† These radiators are all regularly tapped 2 in, and bushed according to the ified in order.

Data for Excelsior Indirect Steam-Radiators

Heating-surface, sq ft	Cold-air supply, sq in	Diameter of duct if round, in	Hot-air flue, sq in	Size for brickwork for hot-air flues, in	Size of register, in	Ratio of 1 to 30, cu ft	Ratio of 1 to 35, cu ft
24	36	6.8	48	4×12	8×8	720	840
36	54	8.3	72	8×12	9×12	1 080	1 260
48	72	9.6	96	8×12	10×14	1 440	1 680
60	90	10.0	120	12×12	12×15	1 800	2 100
72	108	11.7	144	12×12	12×19	2 160	2 520
84	126	12.7	168	12×16	14×22	2 520	2 940
96	144	13.5	192	12×16	14×24	2 880	3 360
108	162	14.4	216	12×20	16×20	3 240	3 780
120	180	15.2	240	12×20	16×24	3 600	4 200
132	198	15.9	264	12×24	20×20	3 960	[illegible]
144	216	16.6	288	12×24	20×24	4 320	[illegible]

l indirect radiators may be used for either water or steam-circula-
ect radiators are generally hung from the ceiling by four iron
ached to the floor-joists and having their lower ends shaped so as
iron pipe or bar-iron on which the radiator rests. The hanger
the return-end of the stack should be slightly lower, from ¼ to ½ in,
thers, so that the water of condensation may have a positive flow
return-connection. The distance from the top of the stack to the
ald be from 10 to 12 in, and the air-space below the stack to the
the casing, from 6 to 8 in. The ceiling over the stack is usually
h galvanized iron or tin. The SUPPLY and RETURN-PIPES should
of ample size. The space required for any quantity of heating-
ny one of the three radiators described above may be readily deter-
neans of the data given. The preceding table will be found useful in
ng the size of air-ducts.

The Boiler

of Heating-Boilers. There are a great many varieties of steam-
se for generating steam for heating purposes besides several types
n the market some years ago and are now practically obsolete. The
ortion of the boilers used at the present time may be classed under
g heads:
ontal tubular boilers.
box boilers.
onal boilers.
Boilers with vertical sections.
Boilers with horizontal sections.

al Tubular Boilers. This boiler has been very extensively used
ating and power and is still preferred by many engineers for heating
ngs or generating steam for hot-blast heating-systems. It is an
pe of boiler, is easily cleaned, and is usually the most economical
arge amount of radiation, say over 2 500 sq ft, and particularly when
used for fuel. The chief objection to its use is that if there is an
is liable to do great damage and may possibly demolish the build-
hance of an explosion, however, is very small indeed,* with low-
eam. Tubular boilers are manufactured in nearly every city of
and can be purchased in every market at a reasonable price. The
d be provided with manholes with strongly reinforced edges, so that
n enter for cleaning. The heads of the boiler above the tubes should
ly braced in order to sustain safely any pressure from the inside.

Domes were formerly placed above the horizontal part of a boiler
the capacity for the storage of steam and to afford a ready means of
dry steam, but they are poor substitutes for properly-designed dry-
e dome is always an element of weakness in a boiler, and many

aim for safety can also be made for the horizontal return tubular boiler.
nat when boilers of this type are properly constructed they do not explode.
mpares the few explosions which occur with the great number of boilers of
ich are manufactured every year and the vastly greater number which are
ge number of them carelessly constructed and carrying a greater pressure
re designed to carry, it is a strong argument in support of this claim, that
ty boilers when *proper care and inspection are given to their construction.*
horizontal return tubular boiler when well designed and carefully constructed
fety boiler, but when compared with water-tube boilers we do not hesitate
more economical." *Edward Kendall & Sons.*

Horizontal Tubular Boilers

Manufactured by Edward Kendall & Sons, Cambridge, Mass.*

Diameter of boiler	Length of boiler		Number of tubes	Diameter of tubes	Length of tubes	Thickness of shell	Heating-surface	Nominal horse-power	Approx. weight of boiler and castings	Square feet of grate-surface†
in	ft	in		in	ft	in	sq ft			
30	6	0	36	2	5	¼	114	7⅗	3 600	5
30	7	0	36	2	6	¼	137	9⅕	3 750	5
30	8	0	36	2	7	¼	160	10⅔	3 900	6
30	9	0	36	2	8	¼	182	12	4 050	8
36	8	0	34	2½	7	¼	189	12½	4 390	8
36	9	0	34	2½	8	¼	216	14½	4 600	8
36	10	0	34	2½	9	¼	243	16	4 810	10
36	11	0	34	2½	10	¼	270	18	5 090	10
36	12	0	34	2½	11	¼	297	20	5 300	12
36	13	0	28	3	12	¼	321	21	5 510	12
42	10	0	45	2½	9	¼	315	21	6 610	12
42	11	0	45	2½	10	¼	350	23	7 030	12
42	12	0	45	2½	11	¼	384	26	7 300	14
42	13	0	45	2½	12	¼	420	28	7 660	14
42	12	0	38	3	11	¼	389	26	7 320	14
42	13	0	38	3	12	¼	425	28	7 680	14
42	14	0	38	3	13	¼	460	31	7 950	16
42	15	0	38	3	14	¼	495	33	8 220	16
48	12	2	69	2½	11	5⁄16	566	38	9 750	18
48	13	2	69	2½	12	5⁄16	617	41	10 150	18
48	15	2	49	3	14	5⁄16	626	42	10 685	18
48	16	2	49	3	15	5⁄16	671	45	11 035	18
48	17	2	49	3	16	5⁄16	716	48	11 485	20
48	17	2	38	3½	16	5⁄16	658	44	12 085	20
48	18	2	38	3½	17	5⁄16	700	47	12 535	20
54	15	2	60	3	14	11⁄32	759	51	14 015	21
54	16	2	72	3	15	11⁄32	954	63	15 074	26
54	17	2	72	3	16	11⁄32	1 018	68	15 584	28
54	18	2	72	3	17	11⁄32	1 082	72	16 094	28
54	17	2	54	3½	16	11⁄32	905	60	15 458	26
54	18	2	54	3½	17	11⁄32	961	64	15 960	26
54	19	2	54	3½	18	11⁄32	1 018	68	16 552	28
60	18	2	92	3	17	⅜	1 364	91	19 000	34
60	18	2	64	3½	17	⅜	1 133	76	18 468	30
60	19	2	64	3½	18	⅜	1 200	80	19 227	32
66	18	2	110	3	17	⅜	1 615	108	22 430	40
66	18	2	82	3½	17	⅜	1 426	95	22 190	36
72	18	2	130	3	17	7⁄16	1 900	127	26 036	48
72	18	2	100	3½	17	7⁄16	1 721	115	25 980	44

---

* Selected from 156 sizes listed by this firm. These boilers are made up to
and 21 ft long.

† For hard coal or coke.

‡ Proportion 6 to 1. The last two columns were added by Mr.

ers claim that a boiler is better without it. For gravity-heating, boilers domes are probably most used. There seem to be no STANDARD TIONS for tubular boilers, as the practice of different makers and engineers omewhat. The proportions given in the accompanying tables, however, ly representative of most of the boilers made, those in the first table esigned for hard coal and those in the second table for soft coal.

soft coal is used for fuel the efficiency of the boiler may be increased asing the grate-area about 20%.

Proportions of Horizontal Tubular Boilers

by the Atlas Engineering Works, Indianapolis, Ind.

are about the standard proportions as used in the Western States for ordinary

Shell†		Mean thickness		Tubes			Heat-ing-sur-face, sq ft	Grate-sur-face, sq ft
Diameter, in	Length, ft	Shell, in	Heads, in	Number	Diameter, in	Length, ft		
36	8	¼	⅜	26	3	8	214	5.8
36	10	¼	⅜	26	3	10	266	8.3
36	12	¼	⅜	26	3	12	318	9.5
40	12	¼	⅜	34	3	12	404	12.0
42	12	¼	7/16	40	3	12	464	12.8
46	12	9/32	7/16	42	3	12	491	14.6
48	12	9/32	7/16	48	3	12	551	15.3
48	14	9/32	7/16	40	3½	14	630	16.0
52	14	5/16	7/16	44	3½	14	693	16.7
54	14	5/16	½	46	3½	14	721	18.0
54	16	5/16	½	40	4	16	817	20.8
60	14	11/32	½	62	3½	14	940	21.5
60	16	11/32	½	52	4	16	1 045	22.2
66	16	⅜	½	64	4	16	1 265	25.0
72	16	7/16	½	82	4	16	1 578	29.5
72	18	7/16	½	82	4	18	1 775	36.5

ill be noticed that these boilers are rated a little higher than the usual standard ft of heating-surface to 1 h.-p.

hese boilers the smoke-box is made of a separate piece, so that the actual length s is 15 in more than length of shell.

of Tubes. In the Eastern States where hard coal is used, 2½-in tubes monly placed in boilers up to 12 ft long, but where soft coal is used for e tubes should not be less than 3 in in diameter even for the smallest while for boilers 16 ft long and over 3½-in or 4-in tubes should be used.

ing of Horizontal Tubular Boilers. Boilers are set with HALF-FRONTS LL FRONTS. With half-front setting, the front end of the boiler projects r more beyond the brickwork and is covered with a cast-iron frame con- two doors for giving access to the flues. With a full front, a cast-iron provided the full width of the boiler and extending from the floor to the the brick setting. Fig. 14 shows the usual method of setting a hori- tubular boiler with full front, and the table opposite it gives the dimen- dicated by the letters, *and the quantities of bricks required.* These will *d useful in showing the boiler-setting on the* foundation-plan of the *and also in estimating the cost of setting.*

Measurements for Setting Tubular Boilers with Full Fronts

Reference-letters in Fig. 14

A	B	C	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	Common brick	Fire-brick
in	ft	in	in	in	in	in	in	in	in	in	in	in	in	in	in	ft in	ft in		
	8	12½	14	30	44	80	18	18	18	8	18	13	35	16	36	5 2	11 7½	5 200	320
	10	12¼	14	36	50	80	18	18	18	8	18	13	35	16	36	5 2	13 7½	5 800	320

rs. A fire-box boiler is a horizontal tubular boiler with a the front end, as in Fig. 15. The fire-box has double walls, being filled with water, so that the fire is entirely surrounded ect being to utilize a greater percentage of the heat generated

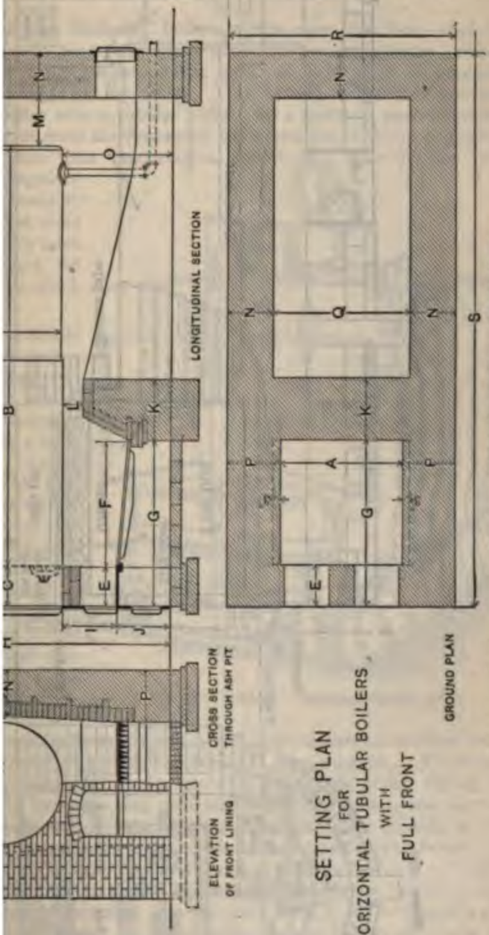
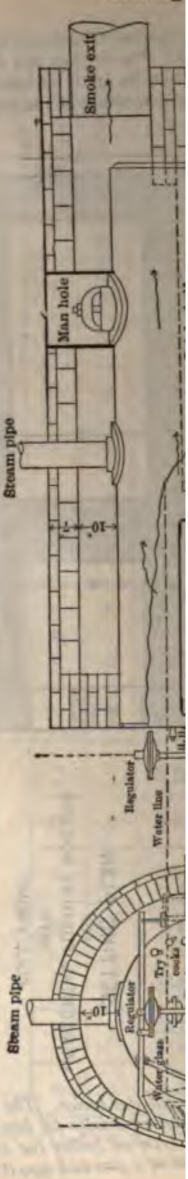

N
M
O
B
L
K
F
G
E
C
I
J
H
P
R
Q
A
S
LONGITUDINAL SECTION
CROSS SECTION THROUGH ASH PIT
ELEVATION OF FRONT LINING
SETTING PLAN FOR HORIZONTAL TUBULAR BOILERS WITH FULL FRONT
GROUND PLAN

Fig. 14. Setting-plan for Horizontal Tubular Boilers with Full Front

n is possible with the *ordinary tubular boiler. The American* *makes a fire-box boiler intended especially for* heating pur- *seem to be a very efficient type of boiler for* buildings *000 ft of direct-steam radiation or* 1 500 to 6 000 ft of hot-


Smoke exit
Man hole
Steam pipe
10
Regulator
Water line
Steam pipe
10
Regulator
Try cocks
Water glass

on, and particularly where hard coal or coke is used for fuel. These
e installed in very low cellars. The danger from explosion with
however, when used for steam-heating is about the same as with
boilers. Fire-box boilers require a brick setting which is shown

Cast-Iron Boilers. Boilers of this class have been used for a
ears, and have very largely taken the place of tubular boilers for
of large private and public buildings, principally on account of
rom dangerous explosions. This, in fact, is the chief advantage
al boiler over a tubular boiler. In a sectional boiler, should an
ur from gross carelessness of the attendant, it would probably be
not more than two sections, and do but little damage to the
any improve-
een made in
so that some
atterns seem
perfect for
rk for which
ded.

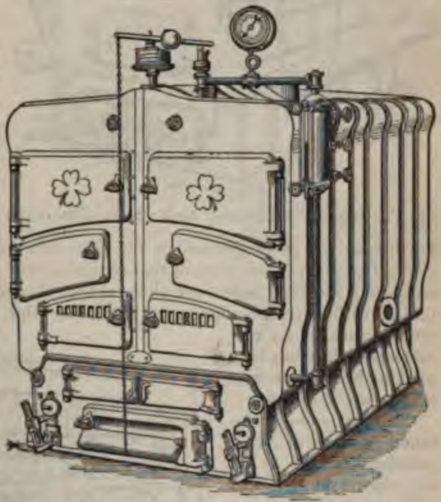


Fig. 16. "Ideal" Sectional Steam-boiler

f Sectional
ere is such a
of small sec-
for house-
is impossible
em in a work
ter. Nearly
ortable pat-
re generally
rizontal sec-
s, with sec-
ne on top of
or buildings
than 400 ft
ation in the
vertical sec-
is to be
ne with hori-
s. The ver-
boiler is made up of a number of cast-iron vertical sections set one
other on a cast-iron base which forms the ash-pit. The sections
together either by means of push-nipples fitting tightly into
ons or by connecting each section to three drums, one above the
on each side near the bottom. The latter type of boiler is desig-
REW-NIPPLE boiler and the former as a PUSH-NIPPLE boiler. The
oiler is the later type and seems to be most in favor with steam-
ords a freer circulation of steam and water than the screw-nipple
ore quickly erected. On the other hand, if any section of a push-
ecomes disabled, the entire boiler must be thrown out of use until
can be put in; while with the screw-nipple boiler, if an inter-
on is disabled it can be disconnected *from the drums* and the
ged, *so that the boiler can be run temporarily*, or even perman-
rge enough, without taking out the disabled section. Fig. 16
al external appearance of push-nipple boilers and Fig. 17 of

the screw-nipple boilers. Fig. 18
Idea sectional water-tube hot-water

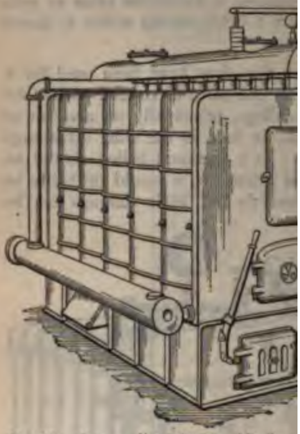
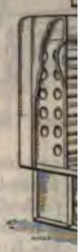


Fig. 17. Gurney "Bright Idea" Boiler,

heating-surface in proportion to
efficient in point of economy of co
sections of sectional boilers are alik
varied within certain limits by
requirements of an economical and satisfactory working-boiler for house-heating are as follows:

(1) They should contain a quantity of water sufficiently large to fill the pipes and radiators with steam to any required pressure WITHOUT LOWERING THE WATER IN THE BOILER TO REQUIRE AN ADDITION WHEN STEAM IS UP; for should the steam go down suddenly, there will be too much water in the boiler. This occurs in boilers made with very small parts or pipes which
have a small capacity at the water-
if it has an automatic water-feeder
but cannot discharge again when th
it is in danger of being spoiled,
STEAM.



F

at a boiler which contains a small amount of water in proportion
-surface will GET UP STEAM quicker than one containing a larger
ater, but the latter will keep steam much better when the fire is
boilers which contain small quantities of water are rapidly chilled
idly heated and must be fired often and regularly.

e-box should be of iron, with a water-space around it, to prevent
the sides and the necessity of repairs to brickwork, which are un-
rick furnaces.

e-box should be DEEP below the fire-door, to admit of a thick fire
ht and thus keep up steam. For large boilers which require the
engineer it is desirable to have a large grate-area and a thin fire;
re requires to be renewed too often to be suitable for a house-

e-box should be SPACIOUS, for the sake of good combustion.

ler should have few parts, and the FLUES and tubes should be LARGE
ical position, so that they will not foul easily, and so that any de-
l to the bottom. For dwellings the writer advises those forms
ich are without tubes, or with but a very few, as the tubes will
e out long before the shell, and if the tubes are not kept clean
smit but a small percentage of heat.

ts should be readily accessible for cleaning and repairs. This is a
greatest importance and economy. When the heating-surfaces
ed with soot and ashes, the economy of the boiler greatly decreases
ts as an insulator and prevents the heat reaching the boiler. It
son that boilers which work well when new are found insufficient
k required of them when they become dirty.

ting-surface should be arranged as nearly as possible at right-angles
ts of heated gases and so break up the currents as to extract the
le heat therefrom.

ld have, if possible, no joints exposed to the direct action of the

ld have a great excess of strength over any legitimate stress, and
onstructed as not to be liable to be stressed by unequal expansion.

uld be durable in construction and not liable to require early

water-space should be divided into sections, so arranged that
ection fail, no general explosion can occur and any destructive
limited to the simple escape of the contents.

uld be proportioned for the work to be done, and be capable of
s full rated capacity with the highest economy.

uld be provided with the very best gauges, safety-valves and
.

should be set so that its water-line will be at least 2 ft below the
tal supply-pipe. The more prominent lines of sectional boilers for
steam or hot-water heating are: The Ideal line, made by the
diator Company; the Bright Idea, made by the Gurney Heater
g Company; the Mercer and Gold, made by the H. B. Smith
nd the Spencer, made by the Spencer Heater Company. Cast-
are very extensively used. Besides being very efficient they also
it of being low in stature, thus fitting them, without constructing
or installation in buildings with low cellars. The American Radi-
y makes *twenty different types of sectional boilers for steam and*
to all kinds of fuel, and fifteen different types of round boilers.

Setting and Covering o
quired for any of the boilers
tight ash-pit about 12 in dee
plastered with a substantial c

Rating of Steam-Boilers.
HORSE-POWER. Strictly spea
OF a steam-boiler, as this is
as boilers are necessary to c
steam-engines has come to b
be discarded. The standard
tennial Exposition in 1876,
evaporation of 30 lb of wate
70 lb gauge-pressure." This
As the amount of water whi
much upon the management
the above standard is a dif
practice the commercial hors
amount of its heating-surfac
ating steam. It is the gener
in horizontal tubular boilers
one horse-power, and most
The rule for computing the
follows, all dimensions bein
cumference of the shell by it
all the tubes by their comm
thirds of the area of both t
tubes, and divide the sum b
heating-surface is equal to th
surface of the shell and both

Steam-Heaters, that is, b
generally rated by the manu
ing-surface they will supply,
made pretty high, so that it
excess of the actual direct ra
50% when they are not cov
figured as equal to 1¾ ft of

Proportioning Radiating
determine the size of boiler
tion, allow 1 sq ft of heating
tion when all mains are cov
are not covered. A large bo
tion in proportion to its hea
piping is not to be included i
that no hard and fast rule
hence in laying out a heati
the advice of an experienced

Amount of Coal Burne
square foot of grate-surface p
and in some cases is very mu
sometimes as small as 3 or 4

Boiler-Trimmings. E
cased steam-gauge, safet

utomatic damper-regulator with connections for operating the
old-air check is also desirable on house-heaters. The best
low-pressure boilers is the single-weighted type; it should be
top of the heater.

Systems of Piping for Steam-Heating

etween Gravity and Non-Gravity Systems. The various
-heating are divided into two general classes, GRAVITY CIR-
ts and NON-GRAVITY SYSTEMS. The former embraces all sys-
e water of condensation from the various radiators returns to
wn weight, that is, by gravity, without the aid of any mechanical
vity systems require some special machinery, such as a pump
return the water to the boiler, or, in some cases, the water of
asted. The kind of boiler used or the character of the radia-
to do with the distinction between the two systems, although
vity systems tubular or power-boilers are generally employed.
ressure steam is carried in the boiler, the non-gravity system
nce, this system is often designated as the high-pressure sys-
y seldom that high-pressure steam is carried into the radiators.
steam is generated for power-purposes, that portion of live
ed for heating is generally passed through a reducing valve, so
in the radiators does not exceed 10 lb per sq in, and if exhaust-
n be mixed with the reduced live steam; otherwise the heating-
the same as a gravity-system, except in returning the water of
he boiler. On the other hand, where low-pressure steam is used
y to place radiators below the water-line in the boiler, a non-
ust be used because the water of condensation must be col-
r receiver and returned to the boiler by a return-trap or pump.
lation the lowest radiation must be at least 4 ft above the
boiler. The same system of piping may be used for both sys-
with the non-gravity systems the return-pipe must terminate
ed below the level of the lowest radiator. (See page 1234.)

Terms Used in Describing Steam and Hot-Water Piping.
n terms used in describing steam or hot-water piping with
t or superintendent should be familiar. The MAIN or DISTRI-
e pipe leaving the boiler and which conveys the steam or hot
ers or branches which supply radiating surfaces. In steam-
is termed the MAIN STEAM-PIPE, and in hot-water heating the
The term SUPPLY-PIPE is sometimes applied to main steam-
not technically correct. The pipes in which the flow takes
diator are called RETURN-PIPES. The MAIN RETURN is the pipe
ith the boiler below the water-line, or, in a non-gravity sys-
h the receiver. The RISERS are those pipes which extend in a
to supply radiators. The vertical pipes in which the flow
called RETURN-RISERS. A RELIEF or DRIP is a small pipe run
in to a return. It must be used at all points where water is
n the main. The PITCH is the inclination given to any pipe
a nearly horizontal direction. The term WATER-LINE is used
ight at which the water will stand in the return-pipes. In a
*ity-system the water-line is practically the level of the water
e term WATER-HAMMER is applied to a very severe concussion
in steam-heating pipes and radiators.* It is caused by cold

water accumulating to such an
pipe, thus forming a vacuum w
water. The water strikes the
and often so as to cause cons
may be prevented by arrangin
of condensation will immediat
TRAP is an upward bend in a
to prevent circulation in the
a small pipe or air-valve for
highest portion of the bend a
the entrapped air.

Systems of Steam-Piping
ity steam-heating which may

(1) **The Mills or Complet**
by J. H. Mills and sometime
this system the main pipe is le
ally to the attic, whence di
risers, which extend to the ba
supply for the radiating-surfa
cases the entire downward
In this system the radiators i
and water of condensation is
main steam-riser. But one c
water of condensation flowing
first floor the piping carries o
equally well adapted for eith
of positiveness of circulation
mended as superior to all oth
for the expansion in the risers
are: (a) The necessity of pla
instead of in the basement, wh
(b) the slightly greater cost c
but as a rule this will be mor
This system is especially reco
factories (see page 1240).

(2) **Ordinary One-Pipe Sy**
system one large steam-main r
radiator or riser is taken off
conveys the water of condens
for dropping the return below
around the basement and con
The steam-main when it leave
and is graded down from the b
densation will flow towards th
there is only one connection m
over the double-pipe system, as
on or shutting off a radiator.
water flow in opposite directio
matic air-valve should be plac
main, above the water-line, to
the radiators. This method
when correctly installed giv

-Pipe System. This system consists in having steam and return-
llar and two connections to each radiator. The steam-main is
om the boiler about ½ in in 10 ft, and is reduced in size as radi-
nections are taken off; at the end it is connected into the return-
water-line. The return-main increases in size as it goes towards
nnections are made to it from risers or radiators. Each radiator
rom a riser or connection taken from the steam-main and empties
through a return-riser or connection, so that there is a complete
ughout the entire system. This system is now confined mainly
ngs and to buildings heated by indirect radiation. INDIRECT
t always have a flow and return-pipe, and when used in build-
the one-pipe system the return-riser must be entered into a re-
w the water-line. The two-pipe system is naturally much more
the one-pipe system, because twice as many radiator-valves
the former and 50 to 75% more piping.

n-Systems of Heating. Several vacuum-systems of heating
market. All of these systems are provided with an exhausting-
ucing a vacuum either on the return or on a pipe-line connecting
es. In most of the vacuum-systems a pump is applied to the
l by its action the pressure is removed from the entire system.
n and most widely used vacuum-system is the Webster, which is
tail later in the second paragraph following.

ystem of Heating.* This is a patented system of exhausting
radiators and piping, so that the steam circulates below or a little
eric pressure. This is accomplished by attaching a patented
h radiator, and at any points where air might possibly collect
and connecting these valves by means of small air-pipes with an
aratus placed in the boiler-room. The valves are so constructed
permit of the passage of air no water can escape through them.
ence between the PAUL SYSTEM and the ORDINARY SINGLE-PIPE
r lies in reducing the pressure by exhausting the air so that the
sened in pipes and radiators. The exhausting-apparatus may be
eam, electricity, gas, or water, water being usually employed
ure systems. The cost of operating the exhausting-apparatus
ure boilers are used need not exceed 3 cts per day for a building
o sq ft of radiation. To install the system the steam-fitter must
alves and exhausting-apparatus from the Automatic Heating
pay a small royalty, the amount depending upon the amount of
building. As by this system better circulation is provided than
ischarges into the rooms through ordinary automatic air-valves
re made more effective; consequently a little less radiation and
re required to do the same work. The cost of installation under
is, therefore, but little if any more than for the ordinary single-
stem, while it is claimed that the system will effect an economy
in the amount of coal required for heating. The system is in
ation in a great many public and private buildings, and the
gents in most of the larger cities from whom further information
d. One great advantage of the system is that people in the
tamper with the air-valves and there is no danger of leaks

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* *See foot-note, page 1234.*

The Webster System * (co
like the Paul system, is a vacu
system, it exhausts all WATER o
are at all times filled with dr
classes of non-gravity heating
Webster radiator-trap is plac
pipe connects each valve with
water of condensation is take
boiler. With this system com
employed, but the radiation sh
ator-trap furnished will depen
be handled and upon the vac
As air is heavier than steam i
The trap may be operated eit
sisting of an aneroidial corr
This system is especially ad
and dry-kilns, and may be
variety of manufacturing proc
details. It has been successfu
the country and in many fact
tion concerning this system m

Return of Water to Boile
As stated on page 1231, wh
than that in the boiler, or wh
the water of condensation mu
below the lowest radiator, and
of some mechanical device.
this purpose. For high-press
machinery or to run the fan i
matically is generally conside
water to the boiler. Where t
turn-trap will generally be
and will return the water as e
The greatest objection to a r
from any cause, it is not as ea
should be placed on or near t
least 2 ft above its water-line.
water-line of the boiler and
hot-blast heating the pump a
stacks and fan.

Fan-System of Warming
in buildings where a large am
the system is the forcing of l

---

* The vacuum-system of heati
try some time in the past seventi
Pa. His plan was to plug up all
and exhaust all air and water fro
plan was an improvement to the
it was not a complete success in i
act only on a portion of the syst
tor's rights and some other pate
valve, which is now used in all t
making their system a success.
duced since.

rooms to be warmed, and necessitates a fan for driving the air. :ssfully operated in connection with hot-air furnaces (see F. E. on Churches and Chapels), but, as a rule, the heat is furnished ım radiation. An ordinary fan-heating and ventilating-plant eam-boiler, one or more stacks of steam-coils, a fan or fans y a small steam-engine or electric-motor, receiver and pump. ls are usually collected in a STACK, over which all of the air for the ed, and from the stack the air is drawn or forced through hot-parts of the building. Direct radiation may also be employed ith this system for warming the halls and corridors or any rooms equire ventilation. This system is especially adapted to the entilating of schools, churches, hospitals and public buildings, tinds of manufacturing plants. To insure successful results, t be laid out by an engineer familiar with this system of heat-nation regarding it may be obtained from the American Blower 3uffalo Forge Company, or the B. F. Sturtevant Company.

s and Valves. The pipe used for conveying steam or hot water ıde exclusively of wrought iron, but at the present time the term PIPE is used merely to distinguish wrought from cast pipe. to mean welded pipe, which is generally made from soft steel.* : IRON PIPE should specify GENUINE WROUGHT-IRON PIPE, for charge is made. Up to the present time the pipe made of steel soft as that of wrought iron, and is often not so well welded and) split. Nevertheless, steel pipe is much more extensively used e wrought-iron pipe, although the latter m-pipe is put on the market in three nesses, STANDARD, EXTRA STRONG and STRONG (see tables of Wrought-iron : to 1276). Each length of pipe as sold ı a collar or coupling (Fig. 19) on one ıread cut on the other. Connections rewing the threaded end of one pipe g on the other. Pipe is sold in random from 16 to 24 ft. With the exception e fittings used for connecting pipes and for giving them any with each other are made of cast and malleable iron. For use s, cast-iron fittings are generally to be preferred to those of (See Carpenter, Heating and Ventilating Buildings.)

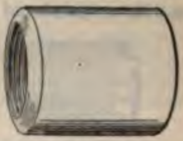
Fig. 19. Wrought-iron Coupling

Joining Pipes. For joining pipes in the same straight line, so ıtinuous pipe from end to end, the coupling, Fig. 19, with right-t in both ends is commonly used. With right-hand couplings :o disconnect the pipe at any place without commencing at the disconnecting the pipe section by section. Reducing-couplings iting pipes of different sizes. To connect two lengths of pipe, ı be disconnected at that point without interfering with other ıds of connections are in use:

Left Couplings. The most common fitting for joining pipes and under. It requires, however, that there shall be room for ne of the pipes sufficient to insert it.

s. These are generally used on pipes up to 1½ or 2 in in diam-desirable to have a joint that may be readily disconnected.

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* See National Tube Company's Handbook.

The union consists of three pie
the pipe and are drawn togeth
thread on one of the pieces, a
appreciable play is required in
connecting radiators, the union

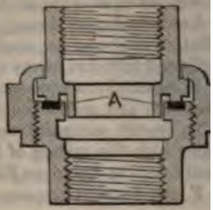

Fig. 20. Lip-union

(3) **Flange-Unions** (Fig. 21).
eter. The two parts of the un
together. A ring of packing m
joint.

Nipples (Fig. 22) are frequ
radiators, and sectional boilers
or with a right thread on one

Fig. 22. Close Nipple

Push-Nipples are made with
make a tight joint by contact
and sectional boilers.

Bushings (Fig. 23) are used

Plugs and Caps. A plug is
covering the end of a pipe.
stock, such as elbows, tees, c
each of these being made in
tion of them may be found i
plies.

Valves and Cocks. Thre
valves (Figs. 24 and 26), ga
and 30). The valve shown in
nated as an ANGLE-VALVE, the
those valves which go on a str
the disk which closes the openi
when open offers less obstructi
reason is largely used on w
gate-valve should not be used

lobe

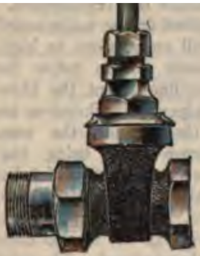

Fig. 25. Brass Gate-valve with Union

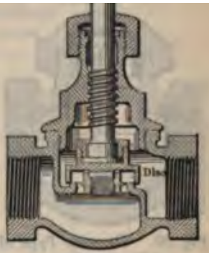

Fig. 26. Section of Disk Globe Valve

on steam-radiators. All valves used for hot water should not e flow when closed. A UNION VALVE is a globe or angle-valve one side of the valve. Globe valves are made for screw, union, ctions, although the latter is commonly used only on large pipes. e-valves for 2-in pipes and under are commonly made with brass

-valve with
pper Disk

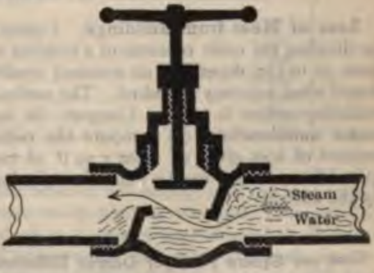

Fig. 28. Showing Obstruction to Flow of Water in Globe Valves

her iron or wooden handles. The larger sizes are commonly bodies. Radiator-valves should have brass bodies and wooden it is desired that radiators shall not be under the control of the he room, valves operated by a key may be used. Hot-water may also be had with pedal attachments so that they may be

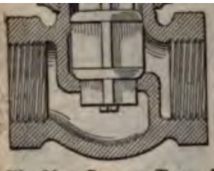

Fig. 29. Common Type of Check-valve

water-line except the blow-off pipes. Check-valves are of three kinds, the most common form being that shown in Fig. 29, which has a valve which slides up and down. The swinging check-valve, Fig. 30, is also commonly employed. The third kind utilizes a ball in place
of the sliding valve for closing the opening. The ball ch
is not much used in steam-fitting. A COCK operates by
plug which has one or two holes bored transversely to its a
is turned so that the hole is in line with the pipe the water
when the plug is turned the water is shut off. Cocks ar
steam-fitting, except on the blow-off pipe.

Fig

Rules for Proportioning Radiating-Surface, and and Hot-Water Mains and Returns

Loss of Heat from Buildings. Formerly radiating-su
by dividing the cubic contents of a building by an empirical
from 50 to 150, depending on assumed conditions. This p
doned when accuracy is desired. The method now generall
radiating-surface is to: (1) Compute the heat required t
under consideration; (2) compute the radiating-surface
amount of heat given off from 1 sq ft of radiation. The
quired for warming may be found by referring to the follow
by the author:

Loss per Square Foot per Degree Difference of Tempera

iple glass, multiply the numbers in the table by $2/(1+n)$, where n is r of thicknesses.

f Heat in British Thermal Units Passing through Walls per Square Area per Degree Difference of Temperature (Fahrenheit) per Hour

Thickness, inches	Single wall		Wall with air-space
	Brick or stone	Wood*	Brick or stone
4	0.43	0.12	0.36
8	0.37	0.065	0.30
12	0.32	0.045	0.25
16	0.28	0.033	0.21
18	0.26	0.031	0.19
20	0.25	0.03	0.18
24	0.24	0.029	0.17
28	0.22	0.027	0.15
32	0.21	0.025	0.13
36	0.20	0.020	0.12
40	0.18	0.018	0.10

periment applies to solid wood; it is evidently of little use when applied to ldings, since these buildings generally present so many opportunities for loss ugh crevices.

r the loss of heat through brick walls given by Recknagel and Reitchel very closely to the values given above. They also give values for ent of heat-loss, expressed in Btu per square foot per hour per degree of temperature, as follows.

ingle window	1.03
ouble window	0.472
ingle skylight	1.090
ouble skylight	0.492
oors	0.410
laster 1.6 to 2.6 in thick	0.615
laster 2.6 to 3.2 in thick	0.492

e preceding paragraphs and tables it will be seen that the heat-trans-rough ordinary walls employed in building-structures is essentially that through glass, and glass transmits approximately 1 Btu per t per degree difference of temperature per hour. This leads to the ormula, in which $(T-t)$ represents the difference between inside and mperatures, G the glass-surface in square feet, and W the exposed e:

$$H = (T-t)\,(G + \tfrac{1}{4}W)$$

formula should be added a correction for the leakage, which is fre-pressed as a function of the cubic contents. For residences it is prac-al to $nC/55$, where C is the cubic contents of the room, and n is the times the air is changed per hour. A more rational method for computing leakage would be to take it as a function of the exposed su which case

$$\text{Heat-loss} = l\,(G + \tfrac{1}{4}W)\,(T - t)$$

in which l has a value from 1⅓ to 2.

Heat from Radiating-Surfaces. The following tables give the am heat in Btu and also the weight of water condensed from various ty radiation on the authority of the author.

Heat-Units Emitted per Hour per Square Foot from Various Surfa
Direct Radiation, Still Air

Difference of temperature, deg. F.	Coefficient or amount per degree difference of temperature				Total per square foot per h		
	Horizontal pipe, diameter				Horizontal pipe, diamete		
	6 in	4 in	2 in	1 in	6 in	4 in	2 in
	Radiator, height				Radiator, height		
	40 in Massed surface	40 in Thin	24 in Massed	12 in Thin	40 in Massed surface	40 in Thin	24 in Massed
80	1.40	1.58	1.67	2.18	112	127	133
90	1.43	1.63	1.72	2.24	128	147	153
100	1.47	1.66	1.76	2.28	147	167	175
110	1.51	1.71	1.80	2.34	166	188	198
120	1.54	1.74	1.84	2.39	184	208	219
130	1.57	1.78	1.88	2.44	203	230	242
140	1.61	1.81	1.91	2.48	223	252	266
150	1.64	1.84	1.94	2.53	244	276	291
160	1.66	1.87	1.97	2.57	265	300	316
170	1.69	1.91	2.02	2.62	286	324	341
180	1.72	1.94	2.05	2.65	307	348	367
190	1.75	1.98	2.09	2.71	330	375	393
200	1.78	2.01	2.12	2.76	356	403	415

* Results divided by 1 000 give approximate weight of steam condensed per

Overhead Steam-Pipes.* When the overhead system of steam- employed, in which system direct radiating-pipes, usually 1¼ in in di are placed in rows overhead suspended upon horizontal racks, the pipes horizontally and side by side around the whole interior of the building, to 3 ft from the wall and from 2 to 4 ft from the ceiling, the amount of pipe required, according to C. J. H. Woodbury, for heating mills, for wh system is deservedly much in vogue, is about 1 ft in length for every 90 space. Of course a great range of difference exists due to the special ch of the operating-machinery in the mill, both in respect to the amount circulated by the machinery and also the aid to warming the room by t tion of the journals. For this system of radiation the Mills system of should be used. (See, also, page 1232.)

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* A. R. Wolff, Stevens Indicator, 1887.

Heat in British Thermal Units and Number of Pounds of
m Condensed per Hour for Different Heating-Surfaces

g-surface	Low pressure. Below 7.5 pounds		High pressure. Above 7.5 pounds	
	Total Btu per hour	Pounds of steam condensed per hour	Total Btu per hour	Pounds of steam condensed per hour
RECT RADIATION				
vertical	260 to 275	0.25 to 0.26	315 to 330	0.30 to 0.31
horizontal	275 to 295	0.26 to 0.28	330 to 350	0.31 to 0.33
	240 to 260	0.23 to 0.25	295 to 315	0.27 to 0.29
ed radiators	150 to 185	0.15 to 0.18	185 to 220	0.17 to 0.21
RECT RADIATION				
wer than 3 ft 3 in	405	0.39	450	0.425
gher than 3 ft 3 in	370	0.35	430	0.415
ed heater, lower n	295	0.28	325	0.306
ed heater, higher n	280	0.27	315	0.298
DIRECT RADIA-ION				
adiator, one row	150 to 165	....	185 to 205	....
adiator, two rows	140 to 155	....	175 to 195	....
adiator, over two	130 to 145	....	165 to 185	....
under 13 ft long,	165 to 185	....	205 to 220	....
over 13 ft long,	185 to 205	....	220 to 240	....
	150 to 165	....	185 to 205	....
ed radiators	85 to 110	....	110 to 140	....
INDIRECT RADIA-ION				
der 3 ft 3 in high	245	....	305	....
er 3 ft 3 in high	235	....	295	....
ed heater, under	185	....	235	....
ed heater, over	175	....	220	....

ron radiators have only about two-thirds the capacity of the American

Size of Steam-Mains and Return-Pipes

H. Babcock gives the following rule for gravity heating-systems eturns, two-pipe system: "The diameter of the steam-mains e boiler should be equal in inches to one-tenth the square root rface, mains included, in square feet." If the mains are cov- be neglected in figuring radiating-surface. For the one-pipe m it will be safe to use one-ninth instead of one-tenth in the less the pipes are very long. It is always better to have the an is necessary rather than too small. Steam-mains should never be less than 1½ in in diameter. The sizes of returns that will prove sati for given sizes of steam-mains are given by the author as follows, no r be less than 1 in diameter:

Diameter steam-pipe, in	Diameter return-pipe, in	Diameter steam-pipe, in	Diameter return-pipe, in
1½	1	5	2½
2	1¼	6	3
2½	1¼	8	3½
3	1½	9	4
3½	1½	10	4½
4	2	12	5

For connecting direct radiators with the SINGLE-PIPE system, the sizes of pipes should be used: For radiators containing 24 sq ft or u pipe; for radiators containing 24 to 60 sq ft, 1¼-in pipe; for radiators ing 60 to 180 sq ft, 1½-in pipe; for radiators containing above 100 s pipe.

For Two-Pipe Work. Radiators containing 48 sq ft and under, 1-i ¾-in return; 50 sq ft to 96 sq ft, 1¼-in supply, 1-in return; above 1½-in supply, 1¼-in return.

For Indirect Heating it will usually be sufficiently accurate to u whose diameter is 1.4 times greater than that for direct heating.

Carrying Capacity of Steam-Pipes

Table for the Capacity of Steam-Pipes 100 Feet in Length with S Returns

By A. R. Wolff

Diameter of supply, in	Diameter of return, in	2-lb pressure		5-lb pressur
		Total heat transmitted, Btu	Radiating-surface, sq ft	Total heat transmitted, Btu
1	1	9 000	36	15 000
1¼	1	18 000	72	30 000
1½	1¼	30 000	120	50 000
2	1½	70 000	280	120 000
2½	2	132 000	528	220 000
3	2½	225 000	900	375 000
3½	2½	330 000	1 320	550 000
4	3	480 000	1 920	800 000
4½	3	690 000	2 760	1 150 000
5	3½	930 000	3 720	1 550 000
6	3½	1 500 000	6 000	2 500 000
7	4	2 250 000	9 000	3 750 000
8	4	3 200 000	12 800	5 400 000
9	4½	4 450 000	17 800	7 500 000
10	5	5 800 000	23 200	9 750 000
12	6	9 250 000	37 000	15 500 000
14	7	13 500 000	54 000	23 000 000
16	8	19 000 000	76 000	32 500 000

n a system of direct hot-water heating should be one pipe-size
the square root of the number of square feet of radiating-surface
for the first story, by 10 for the second story, and by 11 for the
f a building; for indirect hot-water heating multiply above results
HOT-WATER HEATING the return-pipe must have the same diameter
y-pipe, and the capacity of both should be equal to the total capac-
isers. For equalizing hot-water pipes the tables on page 1272 will
ry convenient. The standard tapping for hot-water radiators is as
diators containing 40 sq ft and under, 1 in; above 40 but not ex-
q ft, 1¼ in; above 72 sq. ft, 1½ in.

'o find the size of boiler necessary to supply any given amount of
e page 1230.

Covering of Pipes

s of Insulation. Steam and hot-water mains radiate more heat
n to their surface than do the radiators which they supply, and
eat is needed for warming the space through which the pipes pass,
a very material loss in the consumption of fuel. To reduce this
nimum, it is customary to cover all pipes in unfinished basements
nsulating substance. The saving in fuel effected by a good cov-
ore than pay for its cost in a few seasons.
t insulating substance known is air confined in minute particles or
t heat cannot be removed by convection. No covering can equal
at of perfectly still and stagnant air; and the value of most insulat-
es depends upon the power of holding minute quantities in such a
t circulation cannot take place. The best known insulating sub-
covering of hair-felt, wool, or eiderdown, each of which, however, is
objection that, if kept a long time in a confined atmosphere and at a
of 150° or above, it becomes brittle and partly loses its insulating

ing made by wrapping three or more layers of asbestos paper, each
thick, on the pipe, covering with a layer of hair-felt ¾ in in thick-
rapping the whole with canvas or paper is much used. This cover-
ffective life of about five years on high-pressure steam-pipes and ten
ars on low-temperature pipes. There are a large number of cover-
ly manufactured for use in such a form that they can be easily
emoved if desired. There is a very great difference in the value of

ere are bends or obstructions consider the length of pipe increased as follows:
lbow 40 diameters; globe valve 125 diameters; entrance to tee 60 diam-
btaining the diameter of steam-main to be used in case there is a separate
ly the above results by 0.82. For indirect heating without separate return
re results by 1.4; with separate return use the results in the form given.

these coverings; some of them are very heavy and contain a large mineral matter with little confined air and are very poor insulators. composed entirely of incombustible matter and are nearly as good hair-felt. In general, the value of a covering is inversely proportional weight, the lighter the covering the better its insulating properties; other being equal, the incombustible mineral substances are to be preferred bustible material. The following table gives the results of some of different coverings, which were conducted with great care and ciently large scale to eliminate slight errors of observation. In thickness of the coverings tested was 1 in. Some tests were made coverings of different thicknesses, from which it would appear that the insulating power obtained by increasing the thickness is very slight with the increase in cost. If the material is a good conductor its power is lessened rather than diminished by increasing the thickness certain point."*

Tests Made at Sibley College, Cornell University, on Various Pipe-

Kind of covering
Naked pipe
Two layers asbestos paper, 1-in hair-felt, and canvas cover
Two layers asbestos paper, 1-in hair-felt, canvas cover, wrapped with Manila paper
Two layers asbestos paper, 1 in hair-felt
Hair-felt sectional covering, asbestos-lined
One thickness asbestos board
Four thicknesses asbestos paper
Two layers asbestos paper
Wool-felt, asbestos lined
Wool-felt with air-spaces, asbestos-lined
Wool-felt, plaster-of-Paris lined
Asbestos, molded, mixed with plaster of Paris
Asbestos, felted, pure long fiber
Asbestos and sponge
Asbestos and wool-felt
Magnesia, molded, applied in plastic condition
Magnesia, sectional
Mineral wool, sectional
Rock-wool, fibrous
Rock-wool, felted
Fossil meal, molded, ¾ in thick
Pipe painted with black asphaltum
Pipe painted with light-drab lead paint
Glossy white paint

Sectional Coverings. It may be seen from this table that bestos, and mineral wool are the three materials most valuable for the of steam-pipes, as wool and hair, although being better short-lived on steam-pipes. Wool covering is extensively used,

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* R. C. Carpenter, in Heating and Ventilating Buildings.

. Sectional coverings, molded and formed to fit different sizes
the market, and are used almost exclusively for covering steam
xtent, hot-water pipes. After the sections are applied they are
ed by lacquered brass bands. The fittings, such as elbows and
plastered with plastic asbestos or magnesia and then covered
lied with flour paste.

data, in connection with the preceding table, will enable the
which kind of covering is likely to be the most effective.

Heating. The system of heating by hot water consists of cir-
ter instead of steam in the radiators. The boiler, pipes and
mpletely filled with water,
ulation-pipes being attached
e boiler and the return-pipes
the water in the boiler when
circulates through the pipes
parts with a portion of its
oming colder and heavier,
n through the return-pipes
where it is again heated.
eneral systems of hot-water
open-tank system, and (2)
system, or pressure-system.

open-tank system an open
is connected to the heating-
a way as to receive the
volume of the water due to
eat, and is connected with
y a vent-pipe, so that there
n the tank. Fig. 31 shows
pe of expansion-tank.

pressure-system a similar
ut the vent-pipe is closed
lve, which will open when
aches a certain point, is
erflow-pipe. By increasing
the system, the water may
the temperature of low-pressure steam, and hence less radiating-
ller pipes may be used.

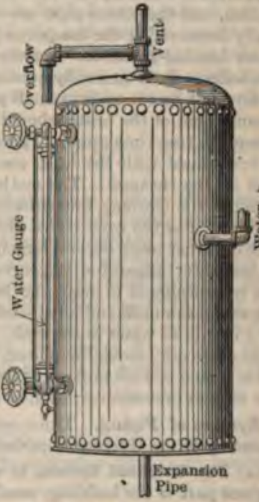

Fig. 31. Expansion-tank

tem is more generally used, although the closed system is used
he closed system is always open to the danger of a serious
the safety-valve becoming inoperative or from the failure of any
ratus. This system cannot be recommended for house-heating.

and other moderate-pressure systems, using a mercury scale
on to the expansion-pipe, are safe. Their highest pressure of
maximum temperature of the radiators from 212° F. to 250° F.
expansion-tank, about the only chance for an explosion is by
the expansion-pipe, either through freezing or by the closing
e pipe. To avoid this, NO STOP OR VALVE should be placed on
ipe, and it should be well protected from frost. It is usually
the supply to one of the radiators in the upper story and the
ways be at least 2 or 3 ft above the highest radiator. The
tank should be somewhat greater than ONE-TWENTIETH of the
ntents of heater, pipes and radiators.

erally forms either the fire-pot or the ash-pit. The suc
water heating-apparatus depends very largely upon th
the boiler. It is generally admitted that in an effici
water must be cut up into small portions, so as to hea
arrangement of the heater should be such that the lea
offered to free circulation. The boiler in which the m
is maintained with the least consumption of fuel is
well as the cheapest. The method employed in conn
facilities for cleaning fire-surfaces are also points that
amined. For the capacity of the various sizes and st
tect or owner must depend largely upon the tables give
A hot-water apparatus is generally filled by connect
the return-pipe at or near the heater. Sometimes a
the expansion-tank and a ball-cock placed on it to insu
be 3 or 4 in of water in the tank. At the lowest point
emptying-cock, should be placed, to empty the system
ratus should be kept FULL OF WATER during the summer
the air and prevents corrosion or oxidation of pipes.

System of Piping. Three systems of hot-water pi
sponding to the three systems described for steam-heat

(1) **The Overhead System,** in which the hot water i
highest part of the building, usually to the attic, and fr
the radiators by return-pipes, exactly as in the Mills
hot water a top and bottom connection is made with
flowing into the radiator at the top and out at the bo
on this system is to have a separate return for the radi
system.

(2) **Two-Pipe System.** This is the system most c
system the mains and distributing pipe have an inclin
heater; the returns are parallel to the main and have
toward the heater, connecting at its lower part. T
from the top of the main and supply one or more radia
from the radiators are connected with the return-pipe i

d in hot-water heating, the return-pipe should be dropped below the
all return-risers should be separately connected with the main

-Pipe System. In this system a single pipe is run around the base-
the one-pipe steam-system, except that the main hot-water pipe rises
oiler; the flow-pipes are taken from the top of the main and the water
ng through the radiators is returned by a separate pipe which is con-
h the bottom of the main. With this system the water in the main
wherever the returns are connected with it, so that the radiators at
l of the system cannot be heated to as high a temperature as those
ive the water as it comes from the boiler. A larger main is required
tem than for system (2). For small jobs, and particularly with boilers
ontal sections, this system may be made to work satisfactorily, but
pe system is always to be preferred. For hot-water heating, special
made which insure a more positive circulation than the ordinary
d in steam-piping.

r Computing Radiating-Surface, diameter of pipes, etc., are given on
, 1241, and 1242.

n of Radiators and Piping. Radiators are more efficient when
lerneath the windows or on inside walls not facing the windows.
radiators are very inefficient and are undesirable from a sanitary
. Indirect radiators should take cold clean air from outdoors and
d so that they can easily be cleaned. The boiler or heater should
as close to the chimney as possible. Steam and hot-water pipes
thoroughly insulated where the placing of them in outside walls or
ld-water pipes cannot be avoided. Supply or return-pipes should
1 in front of cellar or other windows. It is generally cheaper for
of a new building if the location of all the heating-apparatus is care-
ed out on the drawings.

g Radiators may increase their efficiency. Recent tests by John R.
he University of Michigan give the following relative transmission-
idiator-paints:*

Transmission-Value of Radiator-Paints

Kind of surface	Relative transmission
Bare iron surface.................	1.00
Copper bronze....................	0.76
Aluminum bronze................	0.752
Snow-white enamel..............	1.01
No-luster green enamel..........	0.956
Terra-cotta enamel...............	1.038
Maroon glass japan...............	0.997
White-lead paint..................	0.987
White-zinc paint....................	1.01

The heat-transmission depends entirely upon the last coat put on."

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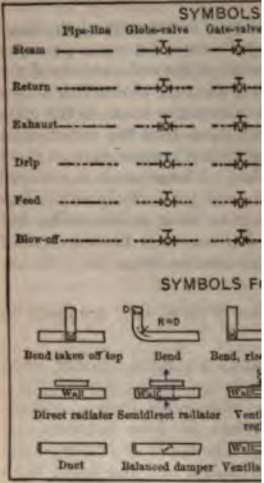
otes on Heating and Ventilation by John R. Allen, published by the Domestic
Company, Chicago, Ill.

Code of Symbols Used by
Indicate Details



Comparative Advantages

(1) **Safety.** An open-tank
sion-tank cannot possibly ex
which is quite unlikely. Wit
sion, although explosions of gr

(2) **Comfort.** There is pro
steam and hot water, if both
become as hot as steam-radiat
not dry, or SCOTCH, the air as
heating must be more healthf
perfectly controlled by either t
by partly closing it; whereas w
or tightly closed. Also, with a
be run at their full capacity, w
out causing noise or in any w
system. A hot-water apparatu
none of the snapping or gurglin

(3) **First Cost.** On an avera
more than a steam apparatus
water apparatus requires nearl
and more expensive fittings.

(4) **Economy in Running.**
given off unless the water is k
off heat with water in the boi

rm weather a hot-water plant will generally keep the rooms com-
less consumption of coal than a steam-heating plant. In very
when the heating-apparatus is worked to its full capacity, there
fference, if any, in the amount of coal consumed for either steam
eating In considering statements as to the economy of differ-
stems, it should be remembered that the economy of any heating-
ends largely on the way in which it is run or upon the person
of the plant.

ges of Hot-Water Heating. About the only objections that
gainst hot-water heating are the increased first cost, danger from
space occupied by radiators, and the fact that a building cannot
warmed by hot water as by steam. It is also more difficult to
circulation in a large hot-water plant than in a large steam
in large buildings and those that are not kept warm all the time
objections are of considerable importance, they do not, as a
d in residences, which are kept at a uniform temperature and in
a size of the radiators is of little consequence. The danger of
y much greater with hot-water circulation than with steam, and
t hot-water indirect radiation must be used with much caution.

For a residence of eight, ten, or twelve rooms probably 90%
re familiar with both steam and hot-water heating would recom-
er. For larger residences and small apartment-houses, about as
recommend steam as hot water, and for still larger buildings,
of the heating engineers would recommend a gravity steam-
er the Webster or the Paul system.

**of Approved Methods for Design of Steam and Hot-Water
Heating.** For convenience of application, the following concise
pproved methods of computation for radiating-surface, dimensions
ate-surface are here given.*

ng Exposed Wall-Surface. Compute area of windows and out-
and one-fourth the exposed wall-surface, ¼ W, for each room.
exposed surface, estimate ceilings and partitions adjacent to un-
as from 30 to 50% exposed. Denote this result by A.

ct Radiation. Compute 2% of the cubic contents of each room.
eating, take once this quantity for second- and third-floor rooms,
ntity for first-floor rooms, three times this quantity for halls.
ns, store-rooms, or bank-rooms, twice this quantity; for large
s, lecture-halls, churches, etc., one-half this quantity under usual
enote this result by B.

iating-Surface, Direct Heating, multiply the sum of the results
ne-fourth for steam-heating; multiply this last quantity by five-
t hot-water heating.

ensions of Piping, Direct Heating, use the tables given. The
for one-pipe systems of steam-heating, commercial sizes of pipes,
h accuracy for dimensions of pipes for hot-water heating, both
d flow-pipe being of dimensions shown in the table. For two-
of steam-heating, use the special table for steam and return-
he table referred to above for steam-pipes less than 3 in, taking
pipe-size smaller than tabulated when above 3 in in diameter.
ses the diameter of return from the special table. In applying

---

* Taken from work on Heating by R. C. Carpenter.

tables, in all cases, find first th
the diameter of submain; anc
sponding in each case to total
The equivalent length of pipe
elbows and bends as explained

(5) **For Radiating-Surface, I**
by the following factors for s
second story, by 0.6; for the t
ply each result as above by fi

(6) **For Dimensions of Pipi**
one-pipe system of steam-hea
steam-heating and for the dia
heating. Take the diameter
special table. Tables to be u

(7) **Size of Air-Flues, Indir**
cross-sectional area for each
For steam-heating, from 1.5 t
sq in for the second story; a
stories. The cold-air flue sup
cross-section of that of the h
have an area equal to that o
to 20% greater for the higher
of flue may be two-thirds as

(8) **Dimensions of Register**
net area not less than from c
the hot-air flue; for ventilat
than the cross-sectional area c

(9) **To Compute the Heatin**
radiating-surface, in which is
from 6 to 8 for the area of h
to 12 for the area of the hea
the AREA OF THE GRATE, divi
by from 120 to 200 for stean
heating.

(10) **To Compute the Area**
surface as explained; if for s
in Professor Carpenter's wor
water heating-system, multipl
then proceed as before.

Hot-Air, Steam an

Choice of a System. A
the methods of heating reside
pose. While it is impossible
believed that the following inf
of heating to be used, in select
properly put in. In deciding
erning conditions are, generall
first cost. When the latter c
ning the apparatus should be
eight or ten rooms and coverin

end hot-air heating by means of a good furnace. For residences
sq ft, a combination hot-air-and-water system is recommended,
ot-water system. For still larger residences, a steam or hot-
ıs should be used.

eating. For warming residences not exceeding 1 200 sq ft of
he author believes a good furnace, properly set and with hot-air
r size, suitably located, will give the best satisfaction, as it is
first cost, easy to manage, costs little for repairs, and furnishes
healthy heat at no greater expense of running than for steam or
he most common defects observed in furnace-heating are over-
air, vitiating of the air by the gases of combustion, and imperfect
the heat. The first two defects may be entirely avoided if
is exercised in the selection and setting up of the furnace and in
e, and the last defect may be reduced to a minimum by a wise
roper proportion of the flues and registers. The cause of the
heating of a great many houses by furnaces is the refusal of the
er to pay
price for a
nace and
kmanship
The same
nd SKIN-
sometimes
n furnace-
itted on a
water ap-
l in most
its work-
Furnace-
e divided
, the PRO-
t and the
f the heat.
pends en-
e furnace,
d-air sup-
d of fuel
e.

ace. In
t-air fur-
a stove
ased with
so as to
amber be-
ater and
air enters
of the chamber, passes over the heated surfaces of the heater,
ted by the hot-air pipes to the various rooms. The external
fire-pot and all portions of the heater which receive heat from
oke are called the RADIATING-SURFACE. As a rule, the furnace
greatest radiating-surface in proportion to the size of the
ive off the most heat for a given amount of fuel consumed.
t of radiating-surface largely *affects the weight of a furnace,*

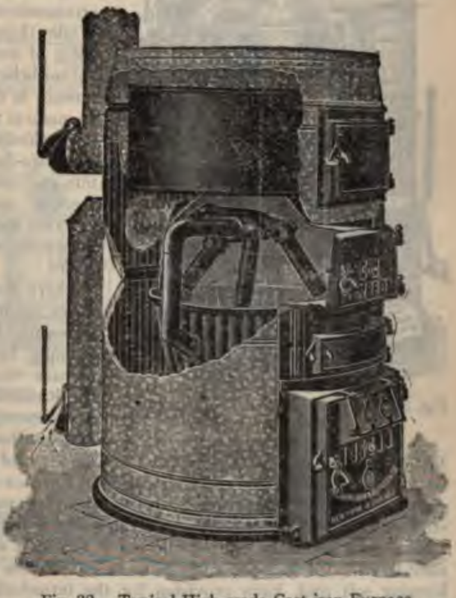


Fig. 32. Typical High-grade Cast-iron Furnace

and the latter in a great measure the selling price, it is ob
best furnaces must cost the most. It is true that one furnace
radiating-surfaces better arranged than another, so as to give of
a less quantity of metal, but it is seldom that a very light furna
if of cast iron, is a good heater. Furnaces should be so designed t
after leaving the combustion-chamber, must travel around the
more times before finding an exit to the chimney. With a chimne
size and topped out well above the roof, it is possible to make th
a long distance and thus obtain great economy of fuel. The be
designed on this principle. Besides having a large radiating-surfa
should have as few joints as possible, and should be arranged so
cleaned. Furnaces are
iron, wrought iron an
used singly or combined
ing-surface above the f
made more cheaply of
than of cast iron, a
arrangements it is just
While there are exce
made of wrought iron
author believes that a h
furnace is the most du
be made as tight. Som
made chiefly of cast iro
or smoke-flues of wrou
into cast-iron sockets.
ment is not generally
the two metals expand
unequally, thus tendin
joints.

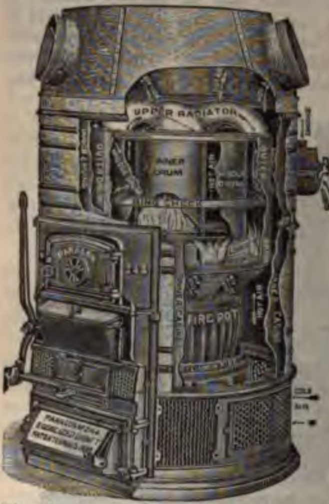


Fig. 33. Typical High-grade Modern Steel-plate Furnace

There are so many
naces manufactured th
impossible to go furth
It may be said, how
furnace shown in Fig.
the Richardson & Boyn
is representative of the
cast-iron furnace, and t
Fig. 33, made by Isaac A. Sheppard & Company, of a modern
nace. Fig. 34, of which the Excelsior Steel Furnace Company a
shows a type of furnace which consists of a plain combustion-c
steel radiator. This radiator is divided by a horizontal part
smoke must circulate entirely around it before it enters the f
nace is intended for soft coal. The more modern furnaces, c
burning soft coal, have provision for the introduction of pre
the fire-box, thereby preventing the formation of soot and ca
combustion and intense heat. The one shown in Fig. 32 is
furnace, and is supplied with air at a high temperature for eith
coal, accelerating and intensifying combustion to a very high deg

In the Twentieth Century furnace the fire-pot contains cells
within the walls of the pot, which admit air at twenty points equ
around the circumference of the same. By reason of this admis
fire burns from the top down and from the circumference tow

ense heat around the outside of the bowl. This furnace can be
essfully with steam-coal. The Thatcher Furnace Company are
ubular furnace that seems to possess considerable merit. The
ding the heater may be of brick or sheet iron. If of brick, it
of two 4-in walls with a space between, the inner wall being
on a circle and the outer one on a square. The BRICK-SET fur-
as common as they formerly were, as they can be cased as well
without occupying so much space in the cellar. When cased
n, the furnace is designated as PORTABLE. Portable furnaces
have two casings with a 1-in space between them. The inner
of black iron, but the outer one should be galvanized. The hot
into the pipes better if the top of the casing is truncated, as in

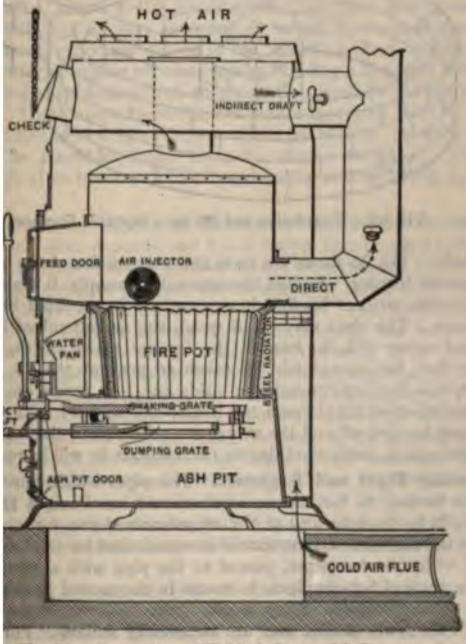


Fig. 34. Connection of Cold-air Duct with Furnace

upply. In a house heated by a furnace, the temperature of the
tained by a constant incoming current of hot air, and it is ABSO-
SARY for satisfactory heating that proper provision be made for
air to the furnace, and on no account should a hot-air furnace be
eing provided with a direct supply of air from outside the building.
is may be best accomplished by putting an opening in the external
th the first-floor joist and as far above the ground as the elevation
will permit. From this opening, which should be covered with

galvanized wire netting of about ¾-in mesh, a duct or flue should be carrie to the air-pit under the furnace, as shown in Fig. 34. The duct may be eithe carried horizontally under the basement-ceiling until near the furnace and the dropped to the air-pit, or it may be carried down against the cellar-wall a thence under the floor to the furnace. The portion of the duct above the flo should be built of well-seasoned matched boards or of galvanized iron. T portion below the floor should be constructed either of stone, brick, or gla tile, and should be tightly cemented. If of brick or stone, the duct should covered with stone slabs with the edges roughly dressed and the joints cemen The air-ducts should not be carried under the floor if the soil is at all damp

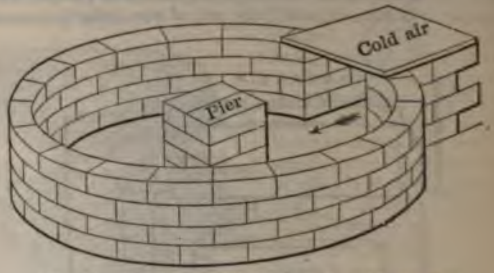


Fig. 35. Foundation and Pit for a Portable Furnace

near any drain. Fig. 35 shows the form and construction of the foundation pit of a portable furnace. Besides the external air-supply, it is also a good to have a smaller air-duct leading from a register in the front hall to the of the furnace. This duct may be of wood, tin, or galvanized iron, and be connected either with the base of the furnace above the floor or feed the outside duct, but care should be taken to prevent the air from blow from the outside duct up through the inside one. An inside duct will prod a better circulation of air through the house, and on very cold nights the side duct may be shut off and the air taken entirely from the front hall, as air from this source, having nothing to contaminate it, will be reasonably p

The Hot-Air Pipes and Registers. The pipes which convey the he air from the furnace to the various rooms should be of bright IX tin for less than 14 in in diameter and of No. 26 galvanized iron for larger sizes. pipes below the basement-ceiling should be round, and for the best work sh be covered with asbestos paper, pasted to the pipe with a specially prep paste. The vertical hot-air pipes, to rooms in the second or third stories, frequently termed STACKS. They usually pass up between the studs of partitions in the lower stories, and are necessarily shallow. For medium and low-cost houses the stacks are usually made 3¾ in deep, of one thickn of tin, and wrapped with asbestos paper pasted on the tin. Two thickn of asbestos paper are but slightly better than one. Double-walled stacks p reduce the fire-hazard to about that of steam-heating. For a better cla buildings double pipes are, or should be, used for the stacks. These stacks an air-space between the outside and inside pipes, affording a circulation of which makes the stacks absolutely safe, thus obviating the necessity of ir in front of the stack. The table on page 1266 gives the sizes and dim of safety double hot-air stacks made by the Excelsior Steel Furnace Com In providing for hot-air stacks, it should be remembered that the les of the pipes largely affects the volume of air conveyed, and ntly a round pipe is always to be preferred to a square one, and to a shallow pipe. In large residences, 5-in or 6-in studding l for partitions, so that thicker pipes may be used. Brick flues used for conveying hot air, as the loss of heat by absorption is very nomical results cannot be obtained. The hot-air registers should le register-boxes made of tin, and the bottom of each stack should BOOT or FOOTING, arranged in such a way as to insure the quick of hot air from the feed-pipe into the stack.

. A hot-air furnace-plant, properly put in, will furnish a good h air, and therefore afford fairly good ventilation, if means are arrying off the foul air in the rooms. The warm air entering a necessity force out an equal quantity of the air already in the re often found in the spaces around the doors and windows, but y sufficient to carry away the air as fast as it would enter if un- eplaces, especially if kept in use, afford excellent ventilation. A ent for obtaining ventilation is by building a large flue in a central using a galvanized-iron smoke-stack, placed in the middle of it, e. The space surrounding the smoke-pipe may then be used for d ducts from different rooms connected with it. Cold-air or exit- ld be placed in rooms likely to become air-bound. The space tuds of an inside wall, if left open to the attic will form an efficient

f Furnace. Upon the location of the furnace the successful house often depends, and it is a matter that requires careful con- s a general rule, the furnace should be located in the basement, lle of the space occupied by the registers, and a little nearer the ch the prevailing winds come in winter. The tendency, in hot- hen the wind is blowing strong in severely cold weather, is for the further side of the house from the wind to be overheated, against the wind to be poorly heated, the registers on the wind- ivering scarcely any hot air. Therefore, to counteract this tend- ace should always be placed a few feet toward the windward side e, provided this does not make the pipes to the general, or family, onger than the others. The height of the basement should be LEADERS, or horizontal hot-air pipes below the basement-ceiling, tch of 1½ in per running foot upward from the furnace. If there on to these pipes, the first-story rooms will be heated with difficulty. e of ten rooms the furnace-room should have a clear height of at

pening. If only one external cold-air supply is used, it should be ne direction from which the prevailing winds come. For build- ed situations it is desirable to have a cold-air supply from the of the building also, and to connect the ducts and furnish each er, so that either duct may be used, according to the direction Cases have been known where the wind blowing from the tion of the cold-air supply has sucked the air from the house through nd cold-air duct, thus actually reversing the natural operation of Two supplies will obviate this possibility.

f Stacks and Registers. To insure the best results, the location acks and registers should be planned out before the work of con- ns, for while the building need not be planned to suit the heating- almost always happens that the setting of the partitions, swinging

that is occasionally blown up through the pipes. I
dirt and many owners object to having their carpets
that it is better to have the registers placed in the
board registers should be used instead of floor-re
Convex registers are to be preferred for walls, as th
do the ordinary flat registers. It sometimes happe
put in an outside wall. When this is the case, the st
wrapped with asbestos paper. Stacks should NOT b
however, when it is possible to place them elsewhere.

Calculations for Size of Furnace, Pipes

Rules for Furnace-Heating.* From the form
rules can be deduced, it being understood that the
equal to the area of windows and doors plus one-fourt
expressed in square feet:

(1) To find area of grate in square inches: Divide
square feet by 1.25 or multiply by 0.8.

(2) To find area of flue for any room in square
glass-surface in square feet by 1.2 for first story, by
for third story.

(3) Make area of vent-flues 0.8 of hot-air flues.

(4) Make area of cold-air box 0.8 of given areas o

(5) Take area of chimney smoke-flue in square inc
grate, with 1 in added to each dimension.

Pipes and Registers. The tables given in variou
the size of pipes and registers vary a great deal and m
able judgment. The table on page 1257 appears to t
as any.

This table gives different sizes of hot-air registers
together with the equivalents of the capacity of the s
from furnace, with an elevation of at least 1 in to the f
riser-pipes (or stacks), and also the cubic feet of s

heated air, but sizes mentioned will produce good results, and, being , will always be found in stock. In planning work arrange to use the ed to. It should always be borne in mind, however, that uniform es not depend so much upon the ACTUAL sizes of the pipes as upon IVE sizes. For example, in a two-story house of eight rooms of HE SAME SIZE and the same amount of wall and glass-area the best sults will be obtained not by using the same size of pipes for all the n if the pipes are of ample capacity, but by carefully proportioning of the pipes according to the exposure, length of the leaders, and f the room in either the first or second story. The registers in the h north and west exposures should be a little nearer the furnace, if han the others, and the pipes to the first story should be larger than ing to the second story. The International Heater Company states in of capacity of hot-air pipe will heat 50 cu ft in stores and 90 hurches when there is but one pipe directly over the furnace.

Table of Capacity of Hot-Air Pipes and Registers

•	Equivalent in round or leader-pipe, in	Equivalent in square or riser-pipe, in	Space in first story same will heat, cu ft	Space in second story same will heat, cu ft	Space in third story same will heat, cu ft
	6	4× 8	400	450	500
	7	4×10	450	500	560
	8	4×10	500	850	880
	8	4×11	800	1 000	1 050
	9	4×12	1 050	1 250	1 320
	9	4×14	1 050	1 350	1 450
	10	4×14	1 500	1 650	1 800
	10	6×10	1 800	2 000	2 200
	10	6×10	1 800	2 000	2 200
	12	6×12	2 200	2 300	2 500
	12	6×12	2 250	2 300	2 500
	12	6×14	2 300	2 600	2 800
	12	6×14	2 300	2 600	2 800
	14	6×16	2 800	3 000	3 200
	14	6×16	2 900	3 000	3 200
	14	8×16	3 000	3 200	3 400
	16	8×18	3 600	4 000	4 250
	16	8×18	3 700	4 000	4 250
	18	10×20	4 800	5 400	5 750
	20	10×24	6 000	7 000	7 450

* Sizes recommended for general use.

r Box. The sectional area of the cold-air box should be equal to hs of the aggregate sectional area of the leaders. The box, or duct, 10 or 12 in deep, for dwellings, and wide enough to give the required rea. It should also always be provided with a damper, so that the y be regulated to the heavy winds and extreme cold weather.

Specifications for Furnace-Work

The following form is given as a guide to architects in preparing the specifications for furnace-work:

SPECIFICATIONS FOR FURNACE-WORK IN RESIDENCE FOR MR. .... TO BE BUILT AT..................................

.................ARCHITECT

Furnace. Furnish and set up complete, where shown on basement-plan, one No. — ——— furnace, portable-pattern, with double casings. Connect the furnace with the chimney with a No. 24 galvanized-iron smoke-pipe of the same size as the collar on the furnace; all bends or turns to be made with three-piece elbows; the pipe to be strongly supported by wire, and to be kept 12 in below the ceiling.

Air-Pit. Excavate for and build a cold-air chamber under the furnace not less than 18 in deep, with 8-in brick walls, laid and plastered with cement, and cement the bottom of the chamber. Build the cold-air duct under cellar floor, where shown on plan, —ft long, 14 in deep in the clear, and—in wide, with sides of hard brick in cement, and with the sides and bottom smoothly plastered with cement. Cover the duct with 3-in flagstones with tight joints, leaving opening of proper size for the wooden box to be built by the carpenter (wooden box should be included in carpenter's specifications).

Hot-Air Pipes. Furnish and properly connect with furnace and register-boxes leaders and stacks of the following sizes, all to be made of bright IX tin, and the stacks are to be double with an air-space. All turns in leaders to be made with three-piece or four-piece elbows, and the stacks to have boots or starters of approved pattern.

Sizes of Pipes and Registers

Hall	12″ leader	No stack	12″×15″ register
Parlor	10″ leader	4″×14″ stack	10″×12″ register
Dining-room	12″ leader	6″×12″ stack	12″×15″ register
Library	10″ leader	4″×14″ stack	10″×12″ register
Chamber No. 1	9″ leader	4″×14″ stack	9″×11″ register
Chamber No. 2	9″ leader	4″×12″ stack	9″×12″ register
Chamber No. 3	8″ leader	4″×10″ stack	8″×10″ register

Registers. All registers are to be of sizes given in the foregoing list, of the Tuttle & Bailey manufacture, japanned, except those in the first story, which are to be electro-bronze-plated. All floor registers are to be set in iron borders corresponding with the registers.

Register-Boxes. All register-boxes to be made double; for first-story boxes the JOISTS ARE TO BE LINED WITH TIN and provided with CEILING-PLATES the full size of register, with plaster-collar attached, so that pipes and boxes can be removed without disturbing the plastering or defacing the ceiling.

Miscellaneous. All horizontal pipes in the basement are to be round, and where they pass through partitions they are to be provided with collars, so that the pipes can be removed without disturbing the plastering. All leaders are to be provided with dampers and tin tags designating the different rooms they supply; and whenever pipes run near woodwork the same is to be properly covered with tin and protected from any danger from fire. The contractor is to remove all rubbish. made by him, clean up all ironwork, and leave the whole apparatus in complete working order, and furnish a poker of proper size.

hen the temperature outside indicates 10 F. below zero. In the
failure of the furnace to do this, the contractor, at his own expense
t unnecessary delay, is either to make the furnace heat said rooms
e another furnace that will heat them.

Hot-Air-and-Water Combination-Furnaces

tion-Furnaces. It is quite difficult, if not impossible, to heat
vering throughout, more than 1 400 sq ft with warm air alone.
of the much larger exposure and the increased length of leaders,
necessary to supplement the warm air with an auxiliary heat which
ied to remote and exposed parts of the house, and which will not
by pressure of wind or long and crooked pipes. For supplying this
at, hot water has been found best adapted as a rule, and a great
OMBINATION-FURNACES are now made which contain provisions for
er which may be carried by pipes to radiators located in those parts
e most difficult to heat by warm air. Such combination-systems
sed with great success, and for heating dwellings of ten average-size
uthor believes it to be the most successful system, as it guaran-
nfortable warming of the house, and, if properly put in, thorough
which cannot be obtained by any system of direct hot-water or
tion. It is claimed that nearly 200 sq ft of hot-water radiation
ined by absorbing the surplus heat which would usually be wasted
air furnace. The construction of the parts for heating the water
ly with different makes of furnaces. Some furnaces have a portion
ot hollow, and the water is heated there; others have a separate
nded over the fire-pot. It is impossible here to consider the relative
e various heaters; the architect should examine the heaters for him-
up their record before specifying any particular make. As a rule,
f the house which should be heated by the hot water are the halls,
and perhaps the rooms on the north or west sides of the house. The
overn the size of the radiators and piping and the manner of install-
entire hot-water plant.

and-Steam Combination. There are also several furnaces which
steam-boiler placed above the fire by means of which a few rooms
ted by direct steam-radiation. Safety-valves are provided so that
ressure cannot exceed 5 lb, and if the directions for running the
re followed, the apparatus is perfectly safe. The steam-combina-
es some advantages over the hot-water combination, and for a large
e author believes that it will give more satisfactory results with in-
nagement.

ation for Hot-Water Heating-Apparatus in a Residence

fication contemplates a complete two-pipe circulating-system, guaran-
in every respect.

Furnish and set up in cellar, where shown on plan, one No. —
— water-boiler, guaranteed free from all flaws and defects. The
set on a substantial foundation of hard brick laid in cement mor-
in by the heating contractor. Furnish and deliver one set of fire-
ting of one poker, one slice-bar and *one fine brush and handle.*

Smoke-pipe. Connect the boiler to the chimney by
made of No. 20 galvanized iron, the diameter of the pi
outlet on the heater.

Trimmings. The boiler to be provided with one e
registering from 80° F. to 250° F. Attach to main flow
one Standard altitude gauge.*

Water-Connections and Blow-off. Feed-water with it
brought within 6 ft of the boiler by the plumber and left
fitting for boiler-connection, which is to be made by this
able cock. Draw-off cock to be placed on lowest poin
fitted for hose-nipple attachment.

Pipes. Furnish and run all necessary flow-pipes and
size, connecting them to radiators with pipes of ample
and rapid flow of hot water to the radiators and easy fl
back to the heater. All connections from risers to radia
floors.

Quality of Materials. All materials used in the constru
are to be the best of their respective kinds, all fittings to
made of the best gray iron with clean-cut threads, and,
and 45° L's are to be used.

Reaming. The ends of all pipes used in the construc
are to be reamed out and all obstructions removed befo
position. All flow-pipes and return-pipes in basement
neat, strong, adjustable hangers, arranged to suit expan
and properly secured to timbers overhead. At all poi
through ceilings, floors, or partitions, the pipes to be
tubes and the holes protected with floor or ceiling-plates.

Expansion-Tank. The expansion-tank to be made of
30 in high and 14 in in diameter, and is to be furnished wit
with brass mountings complete. It is to be placed abo
some suitable place and supported on a proper shelf. Fr
flow-pipe will be run to basement or other suitable place wi
the roof.

Radiators. Furnish and set up the following radiators:

Rooms	Number of radiato
Main hall	1 indirect rad
Sitting-room	1 indirect rad
Library	1 direct radia
Dining-room	1 direct radia
Sitting-room chamber	1 direct radia

of direct surface and 228 sq ft of indirect; total surface, 512
t radiators to be (American Radiator Company's Rococo hot-
in high.

ich radiator will have properly connected to it a nickel-plated
ened and closed with a key.

s. Each direct radiator will be promptly connected to the
with a (Gurney) quick-opening nickel-plated radiator-valve

ion. The indirect radiators to consist of two stacks of the
tor Company's Excelsior) hot-water radiator connected to-
t joints and firmly suspended from the basement-ceiling by
iron hangers. The stacks are to be so piped and hung as to
oiseless and constant flow throughout of the heated water.
enclosed in galvanized-iron chamber with proper inlet for
orresponding outlet for warm air, connected by a galvanized
er in the room which the stack is intended to heat. The regis-
Tuttle & Bailey pattern, electro-bronzed plated, and of the
hall, 12 by 19; sitting-room, 14 by 22 in. Registers to
s and to be set in register-boxes. The pipe connecting the
er is to be so arranged that all fresh air coming in will be
and conveyed, without loss, to its destination. In arranging
re is to be exercised in getting ample space for cold air under
corresponding space for warm air over the stack; unless other-
is space is not to be less than 12 in above and 10 in below the

pe. All flow and return-pipes and fittings in cellar above the
rly covered with 1-in hair-felt neatly sewed up in canvas and
of good white lead, or to be covered with asbestos or magnesia
g with canvas cover and secured by lacquered brass bands.

r. Cover all exposed parts of boiler, except the front, with
½ in thick, neatly applied and troweled smooth.

All work to be done in a neat, substantial and workmanlike
apparatus, when completed, to be thoroughly tested and left
order.

he contractor is to guarantee that the apparatus, when com-
nce with this specification, will be of ample capacity to evenly
erature of 70° F. in the rooms in which radiators are located
temperature is at zero, and that the apparatus throughout
nd rapid circulation when in operation.

Steam-Heating for Residences

uirements. For indirect radiation, steam-heat is generally
er than hot-water heat, and in every way as satisfactory.
sidences, the author would recommend steam-heat, all of the
to be heated by indirect radiation, and only the bath-room,
s the attic and one or two rooms on the north side, which gen-
e dining-room, by direct radiation. For dining-rooms a special
containing a warming-closet, is made. The air-supply to the
hould be very large and provided with a damper, so that the
gulated according to the weather. The same principles apply
ence by steam as in heating any other building, and there is no

by , architect.

Plans. The plans herewith are intended to show
boiler, piping and radiators; the arrangement of the p
to the contractor, subject to the approval of the archi

General Requirement. This contractor is to provid
appliances for the erection and completion of the wo
is to remove all apparatus, refuse and débris from th
leaving the work in a clean, uninjured and perfect co
any description tending to weaken the building structu
without consulting the architects. This contractor is t
the safety and good condition of the work and material
until the completion and acceptance of the same. All
quality, and should at any time improper, imperfect,
faulty workmanship be observed, whether before or af
into the structure, this contractor, upon notice from th
same and substitute good and proper material and wor
in place thereof, in default of which the architect is
means as may be deemed best, and is to deduct the cost
the sum due the contractor under this contract.

System. The heating is to be effected by direct radia
out as shown on plans, and the circulation of the steam
basement-system.

Boiler. This contractor is to build foundation for bo
deep, of common hard brick laid in cement mortar.
boiler of proper size, 12 in deep, cemented, and made
furnish and set up one (C. I. sectional) boiler,
pressure brass-cased steam-gauge, water-gauge, and gl
nation-column, safety-valves and blow-off valves, a
necessary trimmings to complete the boiler,* and a full
ing of one slicing-bar, one hoe, one poker and a cleanin
boiler with 1¼-in of asbestos cement, neatly troweled t

Water-Feed. The plumber is to bring the water-supp ect same to sewer, and vent with 1¼-in pipe to roof. (Catch- omitted in house-heating, a hose being attached to the blow- ing off.)

Contractor is to connect the boiler with the chimney with a made of No. 16 galvanized iron with suitable balance-damper. o be of same size as left for this purpose by maker of boiler.

Risers. The main steam-pipe is to be of ample size to carry radiators attached to the system, and is to be so graded that all ation will flow freely back to the boiler without noise. From ain the various branches are to be taken to radiators and risers, or which are to be so made that no traps are formed, and when ccur they are to have a relief-pipe to carry off all water of con- ntric fittings only to be used on heating-mains where reduced supplies to radiators to be 1 in to 24 sq ft, 1¼ in to 60 sq ft, and e this amount of heating-surface. Radiators on first story to rect to steam-main. All horizontal pipes to be one size larger pipes. All steam-connections from heating-main to radiators on a 45° angle from heating-main and to be one size larger adiator-feeds.

ings. All pipe used throughout is to be of the best quality e of standard weight and thickness, smooth inside, free from d true to shape. All threads to be clean-cut, straight and true. e of the best heavy gray iron, with taper-threads and to be No inferior pipe or fittings will be allowed. All couplings used best make, with recessed ends, except reducers, which are to

piping to be supported by approved expansion-hangers or ceed 10 ft apart. Neat cast-iron floor and ceiling-plates are pipes pass through floors, ceilings and partitions.

rnish direct radiation to the amount enumerated on the plans, n Radiator Company's) make. All 38-in radiators to be the ern. All radiators 14 in high to be (Ætna flue) radiators.

s. The radiators are to be furnished with Jenkins disk union t nickel-plated metal and are to have hard-wood handles.

alves 2 in and under to have brass bodies and iron wheels; over cast iron with brass stem and trimmings and iron wheels; all over to have heavy yokes. All gate-valves to be of (Jenkins)

adiators throughout the entire building are to be furnished tomatic air-valves.

All pipes in the cellar above the floor to be covered with magnesia) sectional covering with canvas cover and secured ss bands.

Bronzing. All radiators and exposed pipes in rooms or halls are inted two coats of best radiator-enamel or bronzed in desired

n completed, the apparatus is to be tested to 20-lb steam- de tight at that pressure, said test to be conducted under the e architect. Fuel for the test to be furnished by the owner, ted the apparatus is to be turned over to the owner in com-

Note. This specification should be accompanied by
the position of all indirect radiation-stacks in the base
to the same, and the size and general position of the mai
and the connections to the radiators. The return-main
below the floor and the steam-main close under the ceili
tion of warm-air registers should be shown on the gen

General Requirements. Boiler, trimmings, and smok
be the same as in preceding specification.

System of Piping. The system of piping throughout
the double-pipe GRAVITY RETURN plan, and all pipes er
size to insure the active delivery of dry steam to the ra
the water of condensation back to the boiler. All supp
branch connection-pipes are to be furnished and erected
are to be of the sizes given and located in the relative
plans. All piping to be graded and properly dripped an
hung in position by means of expansion pipe-hangers.
run at or under basement-floor and protected from m

Stack-Boxes and Flues. The heating of the several ap
plished by means of indirect radiators set in clusters of
the ceiling of the cellar, and the heat from these STACKS
room to be heated by means of tin hot-air pipes set in th
the stack to the room to be heated; each room heated
STACK and to be connected therewith by an independent
of the STACKS of indirect radiators to be enclosed in
vanized-iron chamber and from each STACK a galvani
size is to lead to the opening provided for the same in
is to be placed in the cold-air duct and a tight door in th
radiator. The radiators are to be hung and the cham
will be a space of not less than (12) in above and (10)
the cold-air duct is to connect with the bottom of the
thest from the warm-air outlet.

rary .... 1 stack to contain 72 sq ft
.... 1 stack to contain 36 sq ft
.... 1 stack to contain 24 sq ft

be placed in either the supply or return to radiating-stack,
tomatic air-valve is to be placed on each stack.

actor is to furnish and set in position in each room heated,
register with a deflecting damper, of the size shown on plans.
story to be bronze finish, and all others to be black or white
shall be selected.

d-iron Work. Contractor is to furnish to builder (who is to
iown on plans) all tin wall-pipes and register-boxes for hot
be heated, all to be made of IX tin and of the sizes shown on
is to furnish and erect the galvanized-iron casings for the
specified, with galvanized-iron ducts to the outside openings.
d in a substantial and workmanlike manner.

contractor is to guarantee that the apparatus when com-
nple capacity to maintain an even temperature of 70° F. in
hen the outside temperature is zero, and that the apparatus
lation throughout and be noiseless in operation.

ence-Heating. Much valuable information on residence-
btained from pamphlets published by different manufac-
n are the American Radiator Company, the International
the Gurney Heater Manufacturing Company, Gorton &
ny, Isaac A. Sheppard & Company, and the Excelsior Steel
of Chicago.

The following tables will b
piping and heating-surface o

Tables of Sizes and D

Made by t

Size of stack as listed, in inches	Actual size of outside stack, in inches	Actual size of inside stack, in inches	Area of inside stack, in square inches
4× 8	3⅝× 7⅝	3¼× 7	23
4×10	3⅝× 9⅝	3¼× 9	29
4×11	3⅝×10⅝	3¼×10	32½
4×12	3⅝×11⅝	3¼×11	35
4×14	3⅝×13⅝	3¼×13	41
6×10	5⅜× 9⅝	5¼× 9	47
6×12	5⅜×11⅝	5¼×11	58
6×14	5⅜×13⅝	5¼×13	68
6×16	5⅜×15⅝	5¼×15	79
8×18	7⅜×17⅜	7¼×17	124
10×20	9⅜×19⅜	9¼×19	176
10×24	9¾×23⅝	9¼×23	213

Dimensions of Registers and Borders *

Made by the Tuttle & Bailey Manufacturing Company

Register		Border	
Extreme dimensions, in	Depth open, in	With ribs, floor opening, in	Tin-box size, in
5 5/8 × 7 5/8	1 5/8	....	....
5 1/4 × 9 1/4	2 1/4	....	....
5 1/4 × 11 1/4	2 1/4	....	....
5 1/4 × 14 1/4	2 1/4	....	....
5 1/4 × 16 3/16	2 1/4	....	....
5 1/4 × 19 1/4	2 1/4	....	....
6 3/8 × 9 3/8	2	8 1/8 × 11 1/8	5 5/16 × 8 5/16
6 3/8 × 12 3/8	2	8 1/8 × 14 1/8	5 5/16 × 11 5/16
6 3/8 × 14 3/8	2	8 1/8 × 16 1/8	5 5/16 × 13 5/16
6 3/8 × 17 3/8	2	8 1/8 × 19 1/8	5 5/16 × 16 5/16
7 11/16 × 7 11/16	2 3/8	9 9/16 × 9 9/16	6 9/16 × 6 9/16
7 11/16 × 9 11/16	2 3/8	9 9/16 × 11 9/16	6 9/16 × 8 9/16
7 11/16 × 10 11/16	2 3/8	9 9/16 × 12 9/16	6 9/16 × 9 9/16
7 11/16 × 11 11/16	2 3/8	9 9/16 × 13 9/16	6 9/16 × 10 9/16
7 11/16 × 15 11/16	2 3/8	9 9/16 × 17 9/16	6 9/16 × 14 9/16
7 11/16 × 17 11/16	2 3/8	9 9/16 × 19 9/16	6 9/16 × 16 9/16
7 11/16 × 19 11/16	2 3/8	9 9/16 × 21 9/16	6 9/16 × 18 9/16
7 11/16 × 25 11/16	2 3/8	9 9/16 × 27 9/16	6 9/16 × 24 9/16
8 11/16 × 8 11/16	2 3/4	10 9/16 × 10 9/16	7 9/16 × 7 9/16
8 11/16 × 11 11/16	2 3/4	10 9/16 × 13 9/16	7 9/16 × 10 9/16
9 3/4 × 9 3/4	3	11 7/8 × 11 7/8	8 5/8 × 8 5/8
9 3/4 × 11 3/4	3	11 7/8 × 13 7/8	8 5/8 × 10 5/8
9 3/4 × 13 3/4	3	11 7/8 × 15 7/8	8 5/8 × 12 5/8
9 3/4 × 16 11/16	3	11 7/8 × 18 7/8	8 5/8 × 15 5/8
9 3/4 × 19 3/4	3	11 7/8 × 21 7/8	8 5/8 × 18 5/8
9 3/4 × 22 3/4	3	11 7/8 × 24 7/8	8 5/8 × 21 5/8
9 3/4 × 25 3/4	3	11 7/8 × 27 7/8	8 5/8 × 24 5/8
10 7/8 × 10 7/8	3 1/4	13 1/16 × 13 1/16	9 11/16 × 9 11/16
10 7/8 × 13 7/8	3 1/4	13 1/16 × 16 1/16	9 11/16 × 12 11/16
11 × 15	3 1/4	13 1/16 × 17 1/16	9 11/16 × 13 11/16
10 7/8 × 15 7/8	3 1/4	13 1/16 × 18 1/16	9 11/16 × 14 11/16
10 7/8 × 17 13/16	3 1/4	13 1/16 × 20 1/16	9 11/16 × 16 11/16
10 7/8 × 19 7/8	3 1/4	13 1/16 × 22 1/16	9 11/16 × 18 11/16
10 7/8 × 21 7/8	3 1/4	13 1/16 × 24 1/16	9 11/16 × 20 11/16
11 15/16 × 11 15/16	3 3/8	14 3/16 × 14 3/16	10 11/16 × 10 11/16
11 15/16 × 13 15/16	3 3/8	14 3/16 × 16 3/16	10 11/16 × 12 11/16
12 × 15 15/16	3 3/8	14 3/16 × 18 3/16	10 11/16 × 14 11/16
11 15/16 × 17 7/8	3 3/8	14 3/16 × 20 3/16	10 11/16 × 16 11/16
11 15/16 × 19 7/8	3 3/8	14 3/16 × 22 3/16	10 11/16 × 18 11/16
11 15/16 × 21 7/8	3 3/8	14 3/16 × 24 3/16	10 11/16 × 20 11/16
14 1/16 × 14 1/16	4	16 7/16 × 16 7/16	12 13/16 × 12 13/16
14 1/16 × 16 1/16	4	16 7/16 × 18 7/16	12 13/16 × 14 13/16
13 13/16 × 16 15/16	4	16 7/16 × 19 7/16	12 13/16 × 15 13/16

For special side-wall registers, see page 1270.

These sizes are those most likely to be found in the stock of local dealers.

Dimensions of Registers and Borders. (Continued)

Size of body, in	Register		Border	
	Extreme dimensions, in	Depth open, in	With ribs, floor-opening, in	Tin-box
12×16	14 5/16 ×18	4	16 7/16 ×20 7/16	[illegible]
12×17*	14 5/16 ×19	4	16 7/16 ×21 7/16	[illegible]
12×18	14 5/16 ×20 5/16	4	16 7/16 ×22 7/16	[illegible]
12×19	14 5/16 ×21 5/16	4	16 7/16 ×23 7/16	[illegible]
12×20	14 5/16 ×22	4	16 7/16 ×24 7/16	[illegible]
12×24	14 5/16 ×26	4	16 7/16 ×28 7/16	[illegible]
12×30	14 5/16 ×32	4	....	....
12×36	14 5/16 ×38	4	....	....
14×14	16 5/16 ×16 5/16	4	18 15/16×18 15/16	[illegible]
14×16	16 5/16 ×18 5/16	4	18 15/16×20 15/16	[illegible]
14×18	16⅜ ×20 5/16	4	18 15/16×22 15/16	[illegible]
14×20	16 5/16 ×22 5/16	4	18 15/16×24 15/16	[illegible]
14×22	16⅜ ×24¼	4	18 15/16×26 15/16	[illegible]
15×25	17 11/16×27 15/16	4½	19 13/16×29 15/16	[illegible]
16×16	18 5/16 ×18 5/16	4¼	20⅞ ×20⅞	[illegible]
16×18	18 5/16 ×20 5/16	4¼	20⅞ ×22⅞	[illegible]
16×20	18 5/16 ×22 5/16	4¼	20⅞ ×24⅞	[illegible]
16×22	18 5/16 ×24 5/16	4¼	20⅞ ×26⅞	[illegible]
16×24	18 5/16 ×26 11/16	4¼	20⅞ ×28⅞	[illegible]
16×28	18 5/16 ×30 5/16	4¼	20⅞ ×32⅞	[illegible]
16×32	18 5/16 ×34 5/16	4¼	20⅞ ×36⅞	[illegible]
18×18	20 5/16 ×20 5/16	4¾	22 15/16×22 15/16	[illegible]
18×21	20 5/16 ×23 5/16	4¾	22 15/16×25 15/16	[illegible]
18×24	20 5/16 ×26 5/16	4¾	22 15/16×28 15/16	[illegible]
18×27	20 5/16 ×29 5/16	4¾	22 15/16×31 15/16	[illegible]
18×30	20 5/16 ×32¼	4¾	22 15/16×34 15/16	[illegible]
18×36	20 5/16 ×38¼	4¾	22 15/16×40 15/16	[illegible]
20×20	22⅜ ×22⅜	5½	25⅛ ×25⅛	[illegible]
20×24	22⅜ ×26⅜	5½	25⅛ ×29⅛	[illegible]
20×26*	22 9/16 ×28⅝	5½	25⅛ ×31⅛	[illegible]
21×29	23⅜ ×31⅜	5½	26⅛ ×34⅛	[illegible]
24×24	26 7/16 ×26 7/16	5⅜	29½ ×29½	[illegible]
24×27	26 7/16 ×29⅜	5⅜	29½ ×32½	[illegible]
24×30	26 7/16 ×32⅜	5⅜	29½ ×35½	[illegible]
24×32	26 7/16 ×34⅜	5⅜	29½ ×37½	[illegible]
24×36	26 7/16 ×38⅜	5⅜	29½ ×41½	[illegible]
24×45	26 7/16 ×47⅜	5⅜	29½ ×50½	[illegible]
27×27	29 7/16 ×29 7/16	6	32½ ×32½	[illegible]
27×38	29 7/16 ×40⅜	6½	32½ ×43½	[illegible]
30×30	32⅜ ×32⅜	7¾	35½ ×35½	[illegible]
30×36	32⅜ ×38⅜	7¾	35½ ×41½	[illegible]
30×42	32⅜ ×44⅜	7¾	35½ ×47½	[illegible]

* These sizes are those most likely to be found in the stock of local dealers.

Estimated Capacity of Pipes and Registers

ROUND PIPES

Area, sq in	Diameter of pipe, in	Area, sq in	Diameter of pipe, in	Area, sq in
38	12	113	22	380
50	14	154	24	452
63	16	201	26	531
78	18	254	28	616
95	20	314	30	707

RECTANGULAR PIPES

Area, sq in	Size of pipe, in	Area, sq in	Size of pipe, in	Area, sq in
32	8×20	160	12×18	216
40	8×24	192	12×20	240
48	10×12	120	12×24	288
64	10×15	150	14×14	196
60	10×16	160	14×16	224
72	10×18	180	14×20	280
96	10×20	200	16×16	256
80	12×12	144	16×18	288
96	12×15	180	16×20	320
128	12×16	192	16×24	384

REGISTERS

Capacity, sq in	Size of opening, in	Capacity, sq in	Size of opening, in	Capacity, sq in
40	10×14	93	20×20	267
53	10×16	107	20×24	320
64	12×15	120	20×26	347
80	12×19	152	21×29	406
72	14×22	205	27×27	486
84	15×25	250	27×38	684
80	16×24	256	30×30	600

ROUND REGISTERS

Capacity, sq in	Size of opening, in	Capacity, sq in	Size of opening, in	Capacity, sq in
26	12	75	20	209
33	14	103	24	301
42	16	134	30	471
52	18	169	36	679

T. & B. Special Side-Wall Registers for Shallow Flues and Thin Partitions Use in Baseboards

This register has a single valve, and the front projects 2 in into the room; 13-in flue can be used with 3-in studding, that is, in the first-story rooms

SIZES AND CAPACITIES

Register		Tin flue		Round pipe	
List-size, in	Net air-opening in register-face, sq in	Size, in	Capacity, sq in	Size, in	Capacity, sq in
7×10	47	4 ×10	40	7	[illegible]
7×12	56	4 ×12	48	8	[illegible]
8×13	70	5 ×13	65	9	[illegible]
8×15	80	5 ×15	75	10	[illegible]
10×13	86	6 ×13	78	10	[illegible]
8×17	95	5½×17	93	11	[illegible]

Diameter of Main and Branch Pipes and Square Feet of Coil-Surface They Will Supply in an Open Hot-Water Apparatus when Coils are at Different Altitudes for Direct Radiation or in the Lower Story for Indirect Radiation*

Diameter of pipe, in	Indirect radiation	Direct radiation. Height of coil above bottom of boiler, in feet							
	0	10	20	30	40	50	60	70	80
	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft
¾	49	50	52	53	55	57	59	61	[illegible]
1	87	89	92	95	98	101	103	108	[illegible]
1¼	136	140	144	149	153	158	161	169	[illegible]
1½	196	202	209	214	222	228	235	243	[illegible]
2	349	359	370	380	393	405	413	433	[illegible]
2½	546	561	577	595	613	633	643	678	[illegible]
3	785	807	835	856	888	912	941	974	[illegible]
3½	1 069	1 099	1 132	1 166	1 202	1 241	1 283	1 327	[illegible]
4	1 395	1 436	1 478	1 520	1 571	1 621	1 654	1 733	[illegible]
4½	1 767	1 817	1 871	1 927	1 988	2 052	2 120	2 193	[illegible]
5	2 185	2 244	2 309	2 379	2 454	2 531	2 574	2 713	[illegible]
6	3 140	3 228	3 341	3 424	3 552	3 648	3 763	3 897	[illegible]
7	4 276	4 396	4 528	4 664	4 808	4 964	5 132	5 308	[illegible]
8	5 580	5 744	5 912	6 080	6 284	6 484	6 616	6 932	[illegible]
9	7 068	7 268	7 484	7 708	7 952	8 208	8 482	8 774	[illegible]
10	8 740	8 976	9 236	9 516	9 816	10 124	10 296	10 852	[illegible]

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* F. Schumann.

r of Steam-Supply Pipes and Square Feet of Direct Radiation They Will Supply with 3 Pounds Steam-Pressure *

Distance of radiator from boiler, in feet						
9	64	100	225	324	400	484
sq ft	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft
240	90	72	48	40	36	32
494	185	148	98	82	74	68
863	324	259	172	144	129	118
1 361	510	408	272	226	204	185
2 796	1 049	839	559	466	419	381
4 884	1 831	1 465	977	814	732	666
7 700	2 887	2 310	1 540	1 283	1 155	1 050
11 323	4 246	3 097	2 264	1 887	1 698	1 544
15 819	5 932	4 745	3 164	2 636	2 372	2 157
21 226	7 959	6 368	4 245	3 537	3 184	2 894
27 997	10 361	8 289	5 599	4 666	4 144	3 768
44 230	16 586	13 269	8 846	7 372	6 634	6 031
64 013	24 005	19 204	12 802	10 668	9 602	8 729
89 615	33 605	26 884	17 923	14 936	13 442	12 220
120 275	45 103	36 082	24 055	20 046	18 041	16 401
156 277	58 604	46 883	31 255	26 046	23 441	21 310

* F. Schumann.

Useful Memoranda. Hot-Water Heating

MEASUREMENT OF FLOW-PIPES AND RETURN-PIPES

he purpose of ascertaining the amount of heating-surface in flow-pipes, returnnd risers, the following table is used.

of e.	Square feet in one linear foot	Surface of pipe covering ¾-in hair-felt and canvas		Table of quantity of water contained in 100 linear feet of pipe of different diameters	
		Size of pipe, in	Multiply length by	Diameter of pipe, in	Contents in 100 feet in length, gal
¼	0.27	1	0.79	1	4.50
	0.34	1¼	0.96	1¼	7.75
¼	0.43	1½	1.04	1½	10.59
½	0.50	2	1.09	2	17.43
	0.62	2½	1.20	2½	24.80
½	0.75	3	1.37	3	38.38
	0.92	3½	1.49	3½	51.36
½	1.05	4	1.64	4	66.13
	1.17	....	....	....	....

letermine the surface, multiply the length of pipe by the figures given in le, always pointing off two places.

nple. 500 lin ft of 1-in pipe multiplied by *0.34 equals 170 sq ft.*

Equalization of Pipe-Areas

R. C. Carpenter

Diam. of pipes, in	Number of small pipes required to make area equivalent to larger pipe with allowance for friction									
	½ in	¾ in	1 in	1¼ in	1½ in	2 in	2½ in	3 in	3½ in	4 in
½	1	2.0	3.7	7.6	11.3	19.0	37.0	55.0	80.0	108.0
¾	...	1	1.8	3.7	5.4	9.2	16.7	25.5	39.0	53.0
1	...	...	1	2.0	3.1	5.1	9.3	14.7	27.0	30.0
1¼	...	...	...	1	1.5	2.6	4.5	7.3	10.6	14.7
1½	...	...	...	...	1	1.7	3.1	4.7	7.1	9.3
2	...	...	...	...	...	1	1.83	2.9	4.1	5.8
2½	...	...	...	...	...	...	1	1.7	2.5	3.5
3	...	...	...	...	...	...	....	1	1.5	2.4
3½	...	...	...	...	...	...	....	....	1	1.4
4	...	...	...	...	...	...	....	....	....	1
4½	...	...	...	...	...	...	....	....	....	....
5	...	...	...	...	...	...	....	....	....	....

Equalization of Pipe-Areas

Babcock & Wilcox

Diam. of pipes, in*	Number of smaller pipes equivalent to one larger pipe									
	¾ in	1 in	1½ in	2 in	3 in	4 in	5 in	6 in	7 in	8 in
½	2.27	4.88	15.8	31.7	96.9	205.0	377.0	620.0	918.0	....
¾	1	2.05	6.9	14.0	42.5	90.4	166.0	273.0	405.0	569.0
1	...	1	3.5	6.8	20.9	44.1	81.1	133.0	198.0	278.0
1½	...	...	1	1.3	6.1	13.0	23.8	39.2	58.1	81.7
2	...	...	...	1	3.1	6.5	11.9	19.6	29.0	40.8
2½	...	...	...	...	1.8	3.87	7.1	11.7	17.4	24.4
3	...	...	...	...	1	2.12	3.9	6.4	9.5	13.3
4	...	...	...	...	...	1	1.8	3.0	4.5	6.3
5	...	...	...	...	...	...	1	1.6	2.4	3.4
6	...	...	...	...	...	...	....	1	1.5	2.1
7	...	...	...	...	...	...	....	....	1	1.4
8	...	...	...	...	...	...	....	....	....	1

* Nominal diameters standard steam-pipes and gas-pipes.

Example. To find the number of 2-in pipes to deliver as much fluid as
In the column headed 5, and opposite 2, read 9.9 in upper table and 11.9 in lo
equivalent number of 2-in pipes.

Table of Mains and Branches

American Radiator Company

Main	Branch
1 -in will supply 2	
1¼-in will supply 2	
1½-in will supply 2	
2 -in will supply 2	
2½-in will supply	2 1½-in and 1 1¼-in, or 1 2 -in and 1
3 -in will supply	1 2½-in and 1 2 -in, or 2 2 -in and 1
3½-in will supply	2 2½-in or 1 3 -in, and 1 2 -in or
4 -in will supply	1 3½-in and 1 2½-in, or 2 3 -in or
4½-in will supply	1 3½-in and 1 3 -in, or 1 4 -in and 1
5 -in will supply	1 4 -in and 1 3 -in, or 1 4½-in and 1
6 -in will supply	2 4 -in and 1 3 -in, or 4 3 -in or
7 -in will supply	1 6 -in and 1 4 -in, or 3 4 -in and 1
8 -in will supply	2 6 -in and 1 5 -in, or 5 4 -in and 2

Table of Morris, Tasker & Company, Inc.

External diameter, in	Internal diameter, in	Standard thickness, in	Birmingham wire-gauge, No.	Internal circumference, in	External circumference, in	Internal area		External area		Length of tube per square foot, inside surface,* ft	Length of tube per square foot, outside surface,* ft	Length of tube per square foot, mean surface,* ft	Weight per lineal foot, lb
						sq in	sq ft	sq in	sq ft				
1	0.810	0.095	13	2.545	3.142	0.515	0.0036	0.785	0.0055	4.479	3.820	4.149	0.90
1¼	1.060	0.095	13	3.330	3.927	0.882	0.0061	1.227	0.0085	3.604	3.056	3.330	1.15
1½	1.310	0.095	13	4.115	4.712	1.348	0.0094	1.767	0.0123	2.916	2.547	2.732	1.40
1¾	1.560	0.095	13	4.901	5.498	1.911	0.0133	2.405	0.0167	2.448	2.183	2.316	1.65
2	1.810	0.095	13	5.686	6.283	2.573	0.0179	3.142	0.0218	2.110	1.910	2.010	1.91
2¼	2.060	0.095	13	6.472	7.069	3.333	0.0231	3.976	0.0276	1.854	1.698	1.776	2.16
2½	2.282	0.109	12	7.169	7.854	4.090	0.0284	4.909	0.0341	1.674	1.528	1.601	2.75
2¾	2.532	0.109	12	7.955	8.639	5.035	0.0350	5.940	0.0412	1.508	1.389	1.449	3.04
3	2.782	0.109	12	8.740	9.425	6.079	0.0422	7.069	0.0491	1.373	1.273	1.322	3.33
3¼	3.010	0.120	11	9.456	10.210	7.116	0.0494	8.296	0.0576	1.269	1.175	1.222	3.96
3½	3.260	0.120	11	10.242	10.996	8.347	0.0580	9.621	0.0668	1.172	1.091	1.132	4.28
3¾	3.510	0.120	11	11.027	11.781	9.676	0.0672	11.045	0.0767	1.088	1.019	1.054	4.60
4	3.732	0.134	10	11.724	12.566	10.939	0.0760	12.566	0.0873	1.024	0.955	0.990	5.47
4½	4.232	0.134	10	13.295	14.137	14.066	0.0977	15.904	0.1104	0.903	0.849	0.876	6.17
5	4.704	0.148	9	14.778	15.708	17.379	0.1207	19.635	0.1364	0.812	0.764	0.788	7.58
6	5.670	0.165	8	17.813	18.850	25.250	0.1750	28.274	0.1963	0.674	0.637	0.656	10.16
7	6.670	0.165	8	20.954	21.991	34.942	0.2427	38.485	0.2673	0.573	0.546	0.560	11.90
8	7.670	0.165	8	24.096	25.133	46.204	0.3209	50.266	0.3491	0.498	0.477	0.488	13.65
9	8.640	0.180	7	27.143	28.274	58.630	0.4072	63.617	0.4418	0.442	0.424	0.433	16.76
10	9.594	0.203	6	30.141	31.416	72.292	0.5020	78.540	0.5454	0.398	0.382	0.390	21.00

These tubes are also made in sizes varying by 1 in from 10 to 21 in.

* In estimating the effective steam-heating or boiler-surface of tubes, the surface in contact with air, or gases of combustion (whether internal or external to the tubes), is to be taken.

For heating liquids by steam, superheating steam, or transferring heat from one liquid or one gas to another the mean surface of the tubes is to be taken.

Dimensions of Standard Double-Extra-Strong Pipe

Nominal, in	Actual external diameter, in	Actual internal diameter, in	Thickness, in	Metal-area, sq in	Nominal weight per foot, lb
½	0.840	0.244	0.298	0.507	[illegible]
¾	1.050	0.422	0.314	0.726	[illegible]
1	1.315	0.587	0.364	1.087	[illegible]
1¼	1.660	0.885	0.388	1.549	[illegible]
1½	1.900	1.088	0.406	1.905	[illegible]
2	2.375	1.491	0.442	2.686	[illegible]
2½	2.875	1.755	0.560	4.073	[illegible]
3	3.500	2.284	0.608	5.521	[illegible]
3½	4.000	2.716	0.642	6.772	[illegible]
4	4.500	3.136	0.682	8.180	[illegible]
4½	5.000	3.564	0.718	9.659	[illegible]
5	5.563	4.063	0.750	11.341	[illegible]
6	6.625	4.875	0.875	15.807	[illegible]
7	7.625	5.875	0.875	18.555	[illegible]
8	8.625	6.875	0.875	21.304	[illegible]
LAP-WELDED CASING					
8¼	8.625	8.265	0.180	4.775	[illegible]
8¼	8.625	8.167	0.229	6.040	[illegible]
8¼	8.625	8.082	0.271	7.125	[illegible]
8⅝	9	8.640	0.180	4.987	[illegible]
9⅝	10	9.577	0.211	6.504	[illegible]
10⅝	11	10.594	0.203	6.886	[illegible]
11⅝	12	11.594	0.203	7.526	[illegible]
12½	13	12.457	0.271	10.852	[illegible]
13½	14	13.432	0.284	12.21	[illegible]
14½	15	14.416	0.292	13.49	[illegible]
15½	16	15.416	0.292	14.41	[illegible]

Smoke-Prevention

Classification of Services. O. H. Landreth, in a report to the of Health of Tennessee, published in Engineering News, June 8, the great number of smoke-prevention devices which had been that date as follows:

(1) **Mechanical Stokers.** They effect a material saving in the firing and are efficient smoke-preventers when not pushed above and when the coal does not cake badly. They are rarely susceptible den changes in the rate of firing frequently demanded in service.

(2) **Air-Flues** in side walls, bridge-walls and grate-bars, through when passing is heated. The results are always beneficial, but the difficult to keep clean and in order.

(3) **Coking-Arches,** or spaces in front of the furnace, arched over the fresh coal is coked, both to prevent cooling of the distilled gases them to pass through the hottest part of the furnace just beyond The results are good for normal conditions, but ineffective when the forced. The arches, also, are burned out and injured by working the

or parts of the grate next the furnace-doors reserved
king the coal before it is spread over the grate. These give
he furnace is not forced above its normal capacity. This
d of COKE-FIRING mentioned before.

Furnaces, or furnaces in which the air is supplied to the
te and the products of combustion are taken away from
thus causing a downward draft through the coal, carrying
lown to the highly heated incandescent coal at the bottom
on the grate. This is the most perfect manner of producing
bsolutely smokeless.

draw air in or inject air into the furnace above the grate,
e air and the combustible gases together. A very efficient
ut one liable to be wasteful of fuel by inducing too rapid a

placed in the furnace above the fire to aid in mixing the
vith the air.

aces, of which there are two different styles, neither of
practical.

es which seem to have proved both practical and effective
oke Prevention Company of America and of the American

Ventilation

pplied to a room or building consists in supplying pure air
e out that which has become vitiated. Perfect ventilation
ng an adequate amount of fresh air warmed or cooled to
erature in such a manner that the circulation shall be con-
in all parts of the room or building and at the same time
n of drafts. Ventilation may be broadly classified as NON-
STEMATIC.

Ventilation may be considered as including all ventilation
systematic provision for the admission and escape of the
r for moving the air. All rooms in a building of ordinary
e some ventilation whenever the temperature of the room
hat of the surrounding air. Pettenkofer found that by dif-
walls the air of a room in his house containing 2 650 cu ft
very hour when the difference of exterior temperatures was
same difference of temperature, but with the addition of a
e, the change rose to 3 320 cu ft per hour. With all the
gs about doors and windows pasted up air-tight the change
cu ft per hour.* Even in the case of direct heating, where
supplied for ventilation, there will be a change of air by
in the room which the writer has found practically met by
to from one to three changes in the cubic contents per
ir is introduced into a room, as by ordinary indirect or hot-
al amount of air must be driven from the room, or if air is
, as by the draft in a fireplace, an equal amount of air must
eating by hot-air furnaces and by indirect steam or hot-
generally provide sufficient ventilation for private residences,
cipal rooms are provided with fireplaces or ventilation-flues.

ating and Ventilating Buildings, R. C. Carpenter.

For Systematic Ventilation provision must be made for the admission and expulsion of the air through flues or definite openings and for power for moving the air. The power for moving air for ventilating purposes is obtained in two ways: (1) by expansion due to heating, and (2) by a fan operated by an electric motor or by a steam-engine or gas-engine. Systematic ventilation also presupposes an attempt to admit a definite amount of air, and the first step in any system of ventilation would naturally be to decide upon the amount of air required.

Amount of Air Required for Ventilation. Authorities differ greatly on this point, except that for school-buildings it is generally agreed that 30 cu ft of air per minute for each occupant or 1 800 cu ft per hour should be the standard.* For churches the same amount will give very fair ventilation, but for theaters and auditoriums, which are usually more closely packed and occupied for a longer period, the air-supply should be from 2 000 to 2 500 cu ft per occupant per hour. Hospitals require the greatest air-dilution, and for such buildings an air-supply of from 4 000 to 6 000 cu ft per hour for each bed should be provided, depending upon the character of the cases treated, contagious diseases naturally requiring the greater amount. The quantity of air required is sometimes measured by the number of times the air in a room will need to be changed, but to determine this accurately it is necessary to fall back to the supply per person. Thus, if a schoolroom 27 by 32 ft and 13 ft high contains fifty pupils and a teacher, and 1 800 cu ft per hour is required per person, the total air-supply required per hour will be 1 800 multiplied by 51 or 91 800 cu ft. As the cubic capacity of the room is 11 232 cu ft, the air in the room must be changed [illegible] times per hour to supply 30 cu ft of air per minute to each person. It is seldom that the air in a room is changed oftener than four times per hour by natural ventilation.

Velocity of Entering Air. The velocity of the air through the inlet-registers or grilles should not exceed from 4 to 6 ft per second, the general allowance being 5 ft per second when the inlet is 7 ft or more from the floor.

Estimating Quantity of Air. The quantity of air passing through a flue or opening is measured by multiplying the sectional area of the flue, or the net area of opening in square feet, by the velocity. Thus with a velocity of 5 ft per second the quantity of air passing through an unobstructed opening 1 ft square will equal 5 cu ft per second, 300 cu ft per minute, or 18 000 cu ft per hour. Velocities of air are measured by an instrument called an anemometer.

Location of Inlet and Outlet. W. R. Briggs, of Bridgeport, Conn., many years ago demonstrated quite conclusively that in a rectangular room of moderate size the best results are obtained when the inlet is in an inner wall near the ceiling and the outlet is nearly under the inlet and close to the floor. This is now the general practice in well-designed schoolhouses, and for churches and hospitals when warmed by indirect radiation. For cubical rooms not exceeding 50 ft square the author considers that one inlet is better than several. In the ventilation of theaters the air is sometimes admitted through the ceiling, but more often through the risers of the floor or through specially designed [illegible]

Size of Flues. The size of the flues both for inlet and outlet is determined by the quantity of air to be moved and its velocity, or

$$\text{sectional area of flue in sq ft} = \frac{\text{quantity of air in cubic feet per minute}}{\text{velocity in feet per minute}}$$

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* This amount is required by law in Massachusetts, Utah, Illinois, Indiana, Ohio

locity will depend upon the motive power, the length, size,
e of the flue, the number of turns or offsets, and whether the
horizontal, so that after the theoretical size of the flue has been
actual size will oftentimes need to be increased by an amount
letermined largely by the judgment of the designer. For this
ble practical experience with forced hot-air heating and ventilat-
o lay out the system of flues to the best advantage, and the
lesigning such a plant will do well to secure the assistance of an
n-systems of ventilation the inlet-openings should be of such size
l amount of air may be introduced with a velocity not exceeding
e when the inlet is 5 ft from the floor or 288 ft per minute when
floor or in the walls near the floor. In figuring the size of ordi-
e required area should be increased about 50% to allow for the
1. With light grilles of 1/16 by 5/16-in iron an allowance of 10%
e sufficient. The velocity of the air in VERTICAL FLUES supply-
ould not exceed that through the opening by more than 50%,
locity in the vertical flues of from 500 to 800 ft per minute. The
ugh the connections to the base of the flues should in turn be
t through the flues, while the velocity in the main horizontal
ts should be still higher. "In fact, in schools and churches the
o gradually reduce velocities from the point of leaving the fan to
harge to the rooms. Careful investigation has shown that, every-
l, the velocity in the main horizontal ducts from the fan should
500 ft per minute and preferably 2 000 ft per minute." * The
es or eduction-flues when air is forced into the rooms by a fan
wo-thirds to three-fourths the sectional area of the induction-flues.

**he Quantity of Air, in Cubic Feet, Discharged per Minute Through
of Which the Cross-Sectional Area is One Square Foot**

ERATURE OF THE AIR, 32° F.; ALLOWANCE FOR FRICTION, 50 PER CENT

Excess of temperature of air in flue above that of external air							
10°	15°	20°	25°	30°	50°	100°	150°
34	42	48	54	59	76	108	133
76	94	109	121	134	167	242	298
108	133	153	171	188	242	342	419
133	162	188	210	230	297	419	514
153	188	217	242	265	342	484	593
171	210	242	271	297	383	541	663
188	230	265	297	325	419	593	726
203	248	286	320	351	453	640	784
217	265	306	342	375	484	684	838
230	282	325	363	398	514	724	889
242	297	342	383	419	541	765	937
264	325	373	420	461	594	835	1 006
286	351	405	465	497	643	900	1 115
306	375	453	485	530	688	965	1 185
324	398	460	516	564	727	1 027	1 225
342	420	485	534	594	768	1 080	1 325
383	468	542	604	662	855	1 210	1 480
420	515	596	665	730	942	1 330	1 630

Ventilation and Heating. B. F. Sturtevant Company.

table by 60. While this table does not strictly apply
air into a room it is sufficiently accurate for practical
heating the velocity in flues in feet per minute is likel
story, from 150 to 240; second story, 300; third story
Also in usual conditions of residence-heating the temp
supply-flues averages about 30° F. above the temperatu

Shape and Material of Air-Ducts. The smoother
less will be the friction of the air against it and the gr
air or warm-air flues should always be made of met
iron for flues exceeding 12 in in diameter. Brick flues
or galvanized iron when they convey warm air, not o
but also to lessen the cooling of the air. When brick f
tion lining is not so necessary, although it will mate
Regarding the shape of the flue or duct, round pipes
pipes next best, and rectangular pipes should always b
as possible. With indirect or natural systems of venti
should be supplied by a separate pipe from the heater,
be taken from a steam or hot-water stack. With forced
ventilation all of the air from the heater often enters on
distributing pipes are taken off to supply the risers to
system no branches should leave the mains at right-ang
at an angle of 45° with easy-radius curves in all cases.
made with less than seven pieces or less inside radius
pipe. No 45° elbow should be made of less than four
branch air-duct to flues should have a damper near th
every Y in the system there should be placed a regulati
dampers and fenders should be adjustable. Upon c
these dampers should be adjusted by trial so that e
its proportionate supply of air, and should then be
should be covered with one thickness or several thick
to reduce the loss of heat.

Natural Systems of Heating and Ventilating.
air moves upwards, due to the expansion caused by

rge brick chimney, utilizing the space around the flue for venti-
heat which escapes from the flue will cause a good draft without
st. A draft may also be produced in a vent-flue by means of coils
es placed in the flue just above the air-inlet, or a gas-heater may
for heating the flue. The draft produced by aspiration is not usu-
to draw air any distance through horizontal ducts. Natural sys-
ilation are only effective when used in connection with warming
ventilation in warm weather.

ms of Ventilation. Ventilation by means of a fan may be effected
wo systems: (1) by the PLENUM-SYSTEM, in which the air is forced
n to be warmed and ventilated, or (2) by the EXHAUST-SYSTEM,
air is exhausted from the room.

ust-System. There are many objections to the adoption of this
s a rule it should be avoided when the plenum method can possibly
ith the exhaust system a partial vacuum is created within the
currents and leaks are inward, so that air rushes around doors
, forming unpleasant and sometimes dangerous currents of air.
on of the air in the room is also less thorough when exhausted than
in. The exhaust-system as a rule is used principally for affording
hot weather or for removing disagreeable odors, dust, etc., for
e it is both economical and effective when properly installed.

m-System, or Hot-Blast System, on the other hand, maintains
sure in the room or rooms ventilated and the leakage is outward
ward. By this system the temperature of the air and point of
completely under control. For heating and ventilating theaters,
large schools and churches this is undoubtedly the best system that
yed, and, with the possible exception of churches, is as economical
maintenance as an indirect steam-heating plant, while affording
tilation and greater comfort. This system has also been applied
dings, factories and buildings used for various purposes. The sys-
used in summer as well as in winter, and by providing a cooling-
air may be cooled to any desired temperature. As ordinarily
rced-blast system consists of a heater and fan with flues and ducts
the air to the various apartments as explained on page 1234, and
paratus, with the exception of the vertical flues, is usually located
ent.

ms of ducts are commonly employed, the single-duct and double-
. A typical arrangement of the single-duct system is shown in
e fan is located at one side of the fresh-air chamber, so that air is
t at *A* and is forced through the heater into a warm-air chamber
one large duct with distributing branches is taken off. A by-pass
so that a portion of the air passes under the heater without being
by means of a damper at the mouth of the duct more or less of
may be mixed with the heated air as desired. With this system all
nveyed through the ducts is of the same temperature. With the
system the upper duct conveys only warm air and the under duct
the mixing-damper is placed at the bottom of the riser to each outlet.
em the temperature of the air to each room may be regulated inde-
the others. A modification of the single-duct system is commonly
ting schools in which a large double chamber is located near the
k, one portion being at all times filled with warm air and the other
cool air. From this double chamber a single duct is led to each
he connection is made with the chamber in such a way that either

all warm or all cool air, or any proportion of both, may be admi
duct, the mixing being controlled by a damper operated by a therm
in the room with which the duct connects. This arrangement save
running two pipes, and when a thermostat regulating-apparatus is
trol the dampers is the most practical system. When there are se

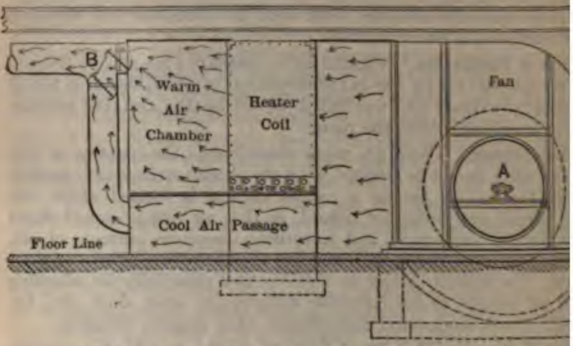

Fig. 36. Type of Single-duct System of Fan-ventilation and Hea

to be warmed and a thermostatic regulating-apparatus is not empl
the mixing-dampers must be operated by hand, the double-duct sy
be employed.

The system shown in Fig. 36 answers very well for warming
auditoriums. Fig. 37 shows the Knowles Mushroom Ventilator wi
sleeve. It is especially adapted for concrete floors. It is made with
5-in opening and a 6-in sleeve. Whe
to be run in warm weather provision
made so that a volume of air equal to
capacity of the fan air may pass
heater. By the arrangement illustra
36 the fan is placed between the heat
cold-air chamber and FORCES the air t
heater. The fan may, however, be
the other side of the heater so as to
through it by exhaustion, at the same
ing the heated air into the ducts. Bot
ments are used, but the former is the
commonly employed. With the
systems of warming and ventilating
chamber of ample size must be provide
to the fan or heater and communica
the outside air by a large duct, the
which should be located as high above the ground as practical con
admit.

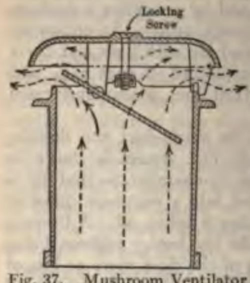

Fig. 37. Mushroom Ventilator with Cast-iron Sleeve. Especially Adapted for Concrete Floors

Forced Blast in Connection with Warm-Air Furnaces. Seve
and churches have been successfully warmed and ventilated by utili
air furnaces of the long tubular pattern to supply the heat and
motor to furnish the power. For churches of moderate size this sys

/e some advantages, especially in economy, over the steam-
lescription of such a system with illustrations may be found in
lder's book on Churches and Chapels, page 148.

r types* of fans are used in connection with the heating and
buildings, the DISK-FAN, the STEEL-PLATE BLOWER or PADDLE-
le MULTIVANE-FAN and the CONE-FAN. The DISK-FAN receives the
and delivers it at the opposite side, the principal motion of the
llel with the axis. This type is only used for exhausting air, and
ised for ventilating single rooms in warm weather. Most of the
sed for ventilating kitchens, restaurants, etc., are of this type.
OR THE PADDLE-WHEEL FAN are the types commonly used with
t systems of heating. It is set in a steel casing and is provided
to eight blades. The tips of the blades are bent backward with
direction of rotation. The MULTIVANE-FAN is provided with
les with the tips bent forward. This fan has great capacity in
the space occupied. A well-known example of this type is the
ade by the American Blower Company, and also the Sturtevant
made by the B. F. Sturtevant Company. The CONE-FAN is a
the paddle-wheel fan with converging entrance-passages. A fan
cy is made by the Buffalo Forge Company. Fans may be driven
g countershaft, from an engine directly connected, or from an
or water-motor. Fans are commonly driven by an electric-motor
e found the most convenient power under most conditions. In
are not used much in warm weather, and in buildings where
up all the year round, a small steam-engine will generally be
al, especially when the exhaust-steam can be used for heating.
some noise, hence they should be located where they will be

Fans. The catalogued capacities of all makes of fans are their
n running light in the open air, not being attached to any ducts
s. These capacities will be reduced from 25 to 50% when so
nding on the length of the ducts and the method of distribution.
acity of fans for forcing air through heating-coils and ducts it is
all the peripheral velocity of the fan-blades equal to the linear
air, and to take ONE-HALF of the theoretical delivery as the actual
e peripheral velocity is obtained by multiplying the revolutions
the circumference of the wheel. Thus a fan 6 ft in diameter run-
tions per minute has a peripheral velocity of 200 × 18.84 = 3 768 ft
Deducting 50% for loss, the actual velocity of the air would be
nute. The discharge-opening in a fan 6 ft in diameter will have
of at least 11.5 sq ft. Multiplying this area by the working
ve 21 666 cu ft per minute as the probable actual discharge of the
ll, of Detroit, who has had extensive engineering experience with
stems, says that the MAXIMUM LIMIT OF SPEED without serious
sure-blower having from six to eight blades, is 2 500 revolutions
l that except in rare cases the blower should run at from 180 to
s per minute. The multivane-type of fan is adapted to run at
hout noise up to 1 500 revolutions per minute. With a disk-fan,
ation only, the velocity should never exceed 900 ft per minute.
bove, the actual capacity when connected with a heating and
em will be reduced from 25 to 50% from the values in the table

---

es of fans are illustrated in *Carpenter's Heating and Ventilating.*

on this page, while the horse-powers, on the other hand, are probably what in excess of those actually required. For further information subject the reader is referred to the catalogues of the various manufa of blowers and to R. C. Carpenter's Heating and Ventilating Buildings. following rule is given by the author for determining the approximate cap of fans of standard proportions.

Approximate Rule for Determining the Capacity of Fans. The cap of fans, expressed in cubic feet of air delivered per minute, is equal to the of the diameter of the fan-wheel in feet multiplied by the number of revol per minute, multiplied by a coefficient having the following approximate

For a fan with a single inlet delivering air without pressure, 0.6; deliveri with pressure of 1 in, 0.5; delivering air with pressure of 1 oz, 0.4. For with double inlets the coefficient should be increased about 50%. For tical purposes of ventilation the capacity of a fan in cubic feet per revol will equal 0.4 the cube of the diameter in feet. Expressed as a formula

$$Q = kD^3n$$

in which Q = cubic feet per minute;
D = diameter of fan-wheel in feet;
n = number of revolutions per minute.

Table of Capacity and Power Required for Steel-Plate Blowers of Var Sizes

With Free Inlet and Outlet

Size, in	Diameter of wheel, in	¼-ounce pressure			½-ounce pressu
		Revolutions	Cubic feet per minute	H.P.	Revolutions	Cubic feet per minute
70	42	214	10 336	0.3	312	14 628
80	48	188	12 584	0.5	265	17 809
90	54	167	16 150	0.7	236	22 856
100	60	150	20 723	0.9	212	29 329
110	66	137	24 548	1.1	193	34 731
120	72	125	30 165	1.3	177	43 676
140	74	107	40 465	1.8	152	57 268
160	96	94	51 344	2.3	133	72 264

Size, in	Diameter of wheel, in	¾-ounce pressure			1-ounce pressu	
		Revolutions	Cubic feet per minute	H.P.	Revolutions	Cubic feet per minute
70	42	377	17 928	1.6	428	20 700
80	48	325	21 827	2.4	367	25 209
90	54	289	28 012	3.7	333	32 343
100	60	260	35 945	4.8	300	41 903
110	66	236	42 579	5.7	273	49 162
120	72	217	52 304	7.0	250	60 392
140	84	186	70 188	9.4	214	81 040
160	96	163	89 057	11.5	152	102 807

ired for Fans. The efficiency of fans under working condi-
ıbout 50%. Consequently the actual power required will be
etical. The theoretical power can be expressed by the follow-

$$\text{H.P.} = \frac{Qh'}{6\,347} = \frac{Qo}{3\,665}$$

.P. = theoretical horse-power;
Q = cubic feet of air per minute;
h' = pressure of air in inches of water;
O = pressure of air in ounces per sq inch.

CHIMNEYS *

chimney is required for two purposes, (1) to create the DRAFT
e proper combustion of the fuel, and (2) to furnish a means of
noxious products of combustion into the atmosphere at such a
ground that they may not prove a nuisance to people living in
he chimney. A good draft is absolutely essential to the satis-
nomical working of either a heating or power-plant, and it is
o build too small a stack and thus limit the boiler-capacity.
s of combustion are poisonous, as in the case of smelters, or
s, tall chimneys enhance the value of surrounding property, if
nore than the cost of the chimney, and should be required by

himneys.† To produce an effective draft in the furnace a
s SIZE and HEIGHT. Each pound of coal burned yields from 15
he volume of which varies with the temperature.

f Gas carried off by a chimney in a given time depends upon
e of chimney, velocity of flow and density of gas. The density
ly as the absolute temperature, while the velocity depends on
the draft and the frictional resistances of the stack, breeching,
d fuel-bed. In designing a stack it must be made of sufficient
ne proper available draft above the fire in the furnace-chamber,
sh this for the operating and furnace-conditions which exist at
nts, it must often be 250 ft high or over.

of Draft at the base of the stack is independent of the size,
unt of friction which exists in a well-designed stack is neglected.
ıpon the difference in weight of the outside and inside columns
ries directly with the product of the height into the difference of
This is usually stated in an equivalent column of water and may
possibly 2 in.

Maximum Draft for any given chimney the heated column
nd the external air 62° F., multiply the height above the grate,
75. The product is the draft-power in inches of water. The
RAFT required varies with the kind and condition of the fuel
ss of the fires. Wood requires the least and fine coal or slack
burn anthracite slack to advantage a forced draft of from 1 to

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n-chief has received valuable assistance and data for the revision of
Henry C. Meyer, author of Steam Power-Plants, Their Design and Con-
bcock & Wilcox Company, the Alphons Custodis Chimney Construc-
. R. Heinicke, The Heine Chimney Company and The M. W. Kellogg

Vilcox Company.

2 in of water is necessary in addition to the chimney-draft. A
CIRCULAR CROSS-SECTIONS is better than one with SQUARE CROSS-
one in which the cross-sectional area is the same throughout its h
than one in which this area decreases with the height. The cross-
at the top, however, may be either larger or smaller without partic

Chimneys for Power-Plants

Size of Chimneys for Power-Plants.* The effective AREA
for a given power varies inversely as the square root of the height
area, in practice, should be greater, because of retardation of v
friction against the walls. On the basis that this is equal to a la
thick over the whole interior surface, and that a commercial h
quires the consumption of an average of 5 lb of coal per hour, we ha
ing formulas:

$$E = \frac{0.3\,H}{\sqrt{h}} = A - 0.6\,\sqrt{A}$$

$$H = 3.33\,E\sqrt{h}$$

$$S = 12\,\sqrt{E} + 4$$

$$D = 13.54\,\sqrt{E} + 4$$

$$h = \left(\frac{0.3\,H}{E}\right)^2$$

in which H = horse-power; h = height of chimney in feet; E =
and A = actual area in square feet; S = side of square chimney
eter in inches of chimney with circular cross-section. The follo
sizes of chimneys was calculated by means of these formulas.

Sizes of Chimneys with Appropriate Horse-Power Capacities

Diameters, in	Effective areas, sq ft	Actual area, sq ft	Height of chimneys									
			50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	125 ft	150 ft	175 ft
			Commercial horse-power capacities of b									
18	0.97	1.77	23	25	27	...	...	...	....	....	....	....
21	1.47	2.41	35	38	41	...	...	...	....	....	....	....
24	2.08	3.14	49	54	58	62	...	...	....	....	....	....
27	2.78	3.98	65	72	78	83	...	...	....	....	....	....
30	3.58	4.91	84	92	100	107	113	...	....	....	....	....
33	4.48	5.94	...	115	125	133	141	...	....	....	....	....
36	5.47	7.07	...	141	152	163	173	182	....	....	....	....
39	6.57	8.30	...	...	183	196	208	219	....	....	....	....
42	7.76	9.62	...	...	216	231	245	258	271	....	....	....
48	10.44	12.57	...	...	...	311	330	348	365	389	....	....
54	13.51	15.90	...	...	...	...	427	449	472	503	551	....
60	16.98	19.64	...	...	...	...	536	565	593	632	692	74
66	20.83	23.76	...	...	...	...	...	694	728	776	849	918
72	25.08	28.27	...	...	...	...	...	835	876	934	1023	1105
78	29.73	33.18	...	...	...	...	...	...	1038	1107	1212	131
84	34.76	38.48	...	...	...	...	...	...	1214	1294	1418	1531
90	40.19	44.18	...	...	...	...	...	...	....	1496	1639	177
96	46.01	50.27	...	...	...	...	...	...	....	....	1876	202

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* These formulas are given by Kent, and are generally accepted a

himneys for House-Heaters. Chimney-flues for heating-appa-
be ample in size and carried as nearly straight as possible from a
he cellar-floor to above the highest projection of the roof. They
dependent, having no connection with other flues or openings and
e same area from top to bottom. A well-jointed tile flue, prefer-
is better than a square brick flue of larger area. The chimney-flue
rried 3 or 4 ft below the smoke-pipe entrance and provided with a
or at the base, tightly fitted, to facilitate the removal of accumu-
nd soot. The size of flues may be calculated from the following

ic of .	Average of direct radiation, steam, sq ft	Tile flues, standard sizes, square or rectangular, outside dimensions, in	Tile flues, standard sizes, round, inside dimensions, in	Brick flues, inside dimensions, in
0 000	200 to 400	8½× 8½	8	8× 8
0 000	450 to 900	8½×13	10	8×12
0 000	1 000 to 1 600	13 ×13	12	12×12
0 000	1 600 to 3 000	18 ×18	16	16×16

table gives the probable size of brick flues for buildings of various
ts. It is approximately correct and may be used in determining
a chimney during preliminary operations for the preparation of
ns; the exact size should be determined after the actual amount of
a building is determined. Indirect radiation should be counted as
han direct radiation and corresponding areas of flues be provided
nount of radiation determines the requisite size of boiler, and there-
a of the flue. No chimney-flue should be less than 8 in in depth,
aller size than the smoke-pipe from the heater. For a KITCHEN
by 8-in tile flue will generally answer, but an 8 by 12-in flue is
FIREPLACES the sectional area of the flue for burning wood or
coal should be from one-tenth to one-eighth that of the fireplace-
a rectangular flue and one-twelfth for a circular flue. For burning
oal the above proportions may be reduced to one-twelfth and one-
spectively. When practicable, chimneys should extend above the
ounding roof, to prevent down-draft caused by eddies. When this is
e a revolving chimney-top will often prevent down-drafts. They
en be avoided by covering the top of the chimney with a stone flag
openings in two parallel sides of the chimney, the sides parallel to
the adjoining roof or building being closed. The WALLS OF THE
be as smooth as possible. Tile flue-lining is preferable. Brick
be either smoothly plastered on the inside with rich lime mortar
should be filled full and struck with the point of the trowel. If
re laid in cement mortar, the author recommends striking the joints
lastering. The walls of attached chimneys with flues not exceeding
nay be 4 in thick for heights of 50 ft. Flues 12 by 12 in and larger
walls 8 in thick to within 10 ft of the top. Aside from strength
thick walls are preferable to thin walls.

of Chimneys. A general rule for the diameter of the base of brick
anding free, approved by many years of practice in England and

the United States, is to make the diameter of the base, or
chimney one-tenth of the height.

Construction of Brick Chimneys. "For chimneys of
100 ft high and upwards, the best form is circular with a st
outside. A circular chimney of this size, in addition to bei
other form, is lighter, stronger, and looks much better
Chimneys of any considerable height are not built up o
from top to bottom nor with a uniformly varying thickn
wall, heaviest of course at the base, is reduced by a series
chimneys of any considerable size should consist of an out
strength to give stability to the structure and an inner
pendent of the outer one. This core is by many enginee
a height of 50 or 60 ft from the base of the chimney, bu
is to run it up the whole height of the chimney. It n
say, a couple feet below the top and the outer shell contr
the core, but the better way is to run it up to about 8 or 1
not contract the outer shell. But under no circumstance
its upper end be built into or connected with the outer st
done in several instances by brick-layers, and the result ha
of the inner core, which lifted the top of the outer stack squ
the brickwork." ¶

Notwithstanding the above, a number of tall brick chim
without an interior wall.

Thickness of Walls. The following is considered as
for the thickness of the outer walls of tall chimneys; the
chimneys is quite complicated and for definite results in an
services of an expert engineer must be employed: For th
top, one brick (8 or 9 in); for the second 25 ft, one and o
on, increasing one-half brick for each 25 ft from the top
inside diameter exceeds 5 ft the top length should be one
the next, two bricks, etc.; if under 3 ft, the top may be on
The batter should be not less than 1 in 36 to give stabili
may be 4 in thick for 25 ft from the top, then 8 or 9 in f
neys of the Edison Station, Brooklyn, each 150 ft high, h
high and one brick thick for the full height, the first 50 ft

Fire-Brick Lining. If a chimney has but one wall it
fire-brick for at least 30 ft, and if it has an inner core, the
of fire-brick for 30 or 50 ft from the bottom. The TOP o
should be protected by a cast-iron cap, which should be
copper wire to an electrical ground driven into damp eart

List of Tall Chimneys

List of Tall Chimneys (Continued)

	Height, ft	Diam. inside at top, ft
Rollox, Scotland, Tenant & Co.	436½	..
nce, Messrs. Musprath Chemical Works	406	..
n Clough Mill, Scotland, Messrs. Crossley's	381	..
C. K. Williams & Co.	375	7
Bolton, England, Dobson & Barlow	367	..
. Y., Eastman Kodak Co. (two chimneys)	366	9 and 13
ook, N. J., Orford Copper Co. (two chimneys)	365	10
s., Fall River Iron Co.	350	11
	359	..
J., Heller & Merz Company	350	8
, Mo., St. Joseph Lead Co.	350	20
, N. J., Clark Thread Co.	335	..
ssia, Germany, Wessenfield & Co.	331	..
Scotland, Gas-Works	329	..
Tenn., Tennessee Copper Co.	325	20
Ind., Indianapolis Traction Co.	320	13
, England, Brook & Son, Fire-clay Works	315	..
England, Adams Soap-Works	312	..
R. I., Rhode Island Suburban Railway Co.	308	16
ity, N. Y., New York Steam Heating Co.	308	15
land, P. Dickon & Son	300	..
ngland, Mitchell Brothers	300	..
h, American Smelting and Refining Co.	300	30
z., American Smelting and Refining Co.	300	25
., Steptoe Valley Traction Co.	300	15
z., Copper Queen Consolidated Mining Co.	300	22
Kent., England, J. C. Johnson	297	..
s., Merrimac Manufacturing Co.	283	..
tland, Camperdown, Linen Works, Cox Bros.	282	..
nce, Schneider & Co.	280	..
ity, Manhattan Railway Co.	278	17
th Lancashire, Darwin & Mostyn Iron Co.	275	..
Pa.	275	..
, Pa., Southern Electric Light and Power Co.	275	18
, Mo., Metropolitan Street Railway Co.	265	16
., Solvay Process Co.	263	12
Eng., Barrow-in-Furness, Hematite Iron Co.	259	..
ngland, Manningham Mills, Lester & Co.	256½	..
N. Y., New York Assay Office	256	7½
N. H., Amoskeag Manufacturing Co.	255	..
rland, England, Hematite Iron Works	250	..
ngland, Story Brothers	250	..
ass., Washington Mills	250	..
., Steptoe Valley Traction Co.	250	13

d by the Alphons Custodis Chimney Construction Company, New York

d by H. R. Heinicke, Incorporated, New York City.
d by The Heine Chimney Company, Chicago, Ill.
d by The M. W. Kellogg Company, New York City.

List of Tall C

*Kansas City, Mo., Armour Packin
*Boston, Mass., Edison Electric Illu
*New York City, Jacob Ruppert Ic
†Rockford, Del., Jos. Bancroft & So
†Niagara Falls, N. Y., Acker Proces
§Detroit, Mich., Solvay Process Co.
§Baptist's Island, P. Q., Wayaganac
Cheshire, England, Connah's Quay
*Kansas City, Mo., Consolidated E
Bradford, England, Newland's Mil
*Cleveland, Ohio, Cleveland City R
Boston Navy Yard, Mass. . . . . . .
†Chicago, Ill., Sears-Roebuck Co. . .
Providence, R. I., Narragansett E.
†Dayton, Ohio, National Cash Regi
*Millinocket, Me., Great Northern
*Weehawken, N. J., N. Y. C. & H.
Lawrence, Mass., Pacific Mills. . . .
Hardwich, Dovercourt, England,
†Indianapolis, Ind., Indianapolis W
Lowell, Mass., Fremont & Suffolk
‡Milwaukee, Wis., Pabst Brewing C
‡Wheaton, Ill., Elgin, Aurora and C
‡Canton, Ill., Parlin & Orendorff Co
†So. Bethlehem, Pa., Didier March
†New Orleans, La., American Sugar
†Grand Rapids, Mich., Waterworks
†Lowell, Mass., Hamilton Manufact
*Edgewater, N. J., New York Gluco
*Washington, D. C., St. Elizabeth's
Woolwich Arsenal, England, Shell-
Northfleet, England, F. C. Gostling
||Elizabethport, N. J., Plymouth Co
Ivorydale, Ohio, Procter & Gambl
Lawrence, Mass., The Lower Pacif
Philadelphia, Pa., The Fidelity Ins
Dewsbury, England, Olroyd & Son
Lanarkshire, England, Coltness Iro
Wilmington, Del., City Water-Wor
Philadelphia, Pa., Finley & Schlect
Camden, N. J., Highland Mill, S. B
Ironton, Ohio, Etna Iron-Works. .
Shamokin, Pa., John M. Sharpless

* Constructed by the Alphons Custod
City.

† Constructed by H. R. Heinicke, In

‡ Constructed by the Heine Chimne

§ Constructed by The M. W. Kell

|| Reinforced concrete.

List of Tall Chimneys (Continued)

	Height, ft	Diam, inside at top, ft
., Hartman General Electric Co.	200	..
, Passaic Print-Works	200	..
Minn., General Electric Co.	200	13½
ich., Bryant Paper Co.	200	10
United States Brewing Co.	200	8
orged Steel Wheel Co.	200	12
, Ia., Sinclair & Co., Ltd.,	200	10
., Packard Motor Car Co.	200	12
Western Glucose Co.	200	10
hem, Pa., Didier March Co.	200	10
ce, Schneider & Co.	197	..
, N. J., Clark's Thread-Mill	192	..
hio, Ohio Rolling Mill Co.	190	..
England, Stanton Iron Co.	190	..
ffield, England, Fox & Co.	186	..
Pa., John Lang Paper-Mills	181	..
J., Lombard, Ayres & Co. Oil Refinery	180	..
an., Winnipeg Agricultural College	150	6

structed by H. R. Heinicke, Incorporated, New York City.
structed by The Heine Chimney Company, Chicago, Ill.

k Chimneys. These chimneys are built with special blocks
the circular and radial lines of each section of the chimney so
ed brickwork has joints of an even thickness throughout and a
th surface. The blocks being much larger than common bricks,
from one-third to one-half as many joints. Radial-brick chim-
s circular in plan above the base. The best form of base is octag-
ection so as to permit the breeching to enter the chimney at a
d at the same time comply best with the rules of stability. Ex-
ical-works, refineries, furnaces, etc., radial-brick chimneys are
ngle shell, a lining only being provided in the immediate vicinity
rance. All radial bricks are perforated vertically and this insures
ing and allows the mortar to enter the perforations, thus forming
horage.

ks for chimney-construction have been used extensively in Eng-
, France and Russia since 1870. They were not introduced into
however, until 1898. About forty-five years ago (1869 or 1870)
dis, of Düsseldorf, Germany, originated a method of building tall
erforated radial blocks, made from selected clays and burned at a
perature, and in 1898 an American company* was formed for the
cting chimneys by this method of construction. Since that time
hrough various agencies has built more than six thousand chimneys
the world. The tallest and largest chimney in the world (1913),
d 50 ft in internal diameter at the top, was built by this company
Boston & Montana Consolidated Copper and Silver Mining Com-
Falls, Mont.

Heinicke,† of Chemnitz, Germany, builder of the 460-ft stack at

ns Custodis Chimney Construction Company, New York City.
Heinicke, Incorporated, New York City.

Halsbrücke, Germany, has empl
chimney. This firm has erected
various diameters in all parts of t
has done much to make chimne
brücke is a very remarkable one
of 460 ft, the diameter at the top
Falls, Mont., has a diameter of 5

The Heine Chimney Company
The essential difference in the m
from those of the other chimney-
pany uses perforated, INTERLOC
interlocking-feature has an adva
as a preventive of deep weatheri
to this it is claimed that the cir
of this type of bricks is consider
or corrugated bricks. The perfor
those of some of the other const
beds with SHOVED joints. Thes
in them thus forming PINS which
to withstand the stresses due to
the flue-gases. In walls more t
English bond, that is, with alter
pany advocates this method of
ordinary straight-sided common
chimneys constructed by the H
the St. Joseph Lead Company's
this chimney is 350 ft and the insi

The W. M. Kellogg Company
chimneys for power-plants, chemi
important chimneys put up by t
already given. Some of the detai
companies mentioned. One of t
the corrugations on their bricks.
deep and are placed along the ve
The adhesion between the bricks
It is claimed that tests made sh
corrugations it is not considere
chimneys to prevent the develop
work has sometimes been inserte
is claimed that this design is some
constructors, this company holdi
of less than 25 lb per sq ft of pr
erected by this company may be
Ariz., erected for the Copper Qu

Reinforced-Concrete Chimn
crete came into quite extensive
some were built before that time
them in use. While the metho
greatly in later years those used
prove that many of those first b
instances, no doubt, efforts tow

---

* The Heine C
† The W. M.

to the general construction of reinforced-concrete chimneys.

: is an outer main shell, a lining or inner shell and the base.

:hickness of the outer shell varies from 4 to 6 in at the top and in at the bottom, the height and diameter of the structure affect-ails.

hickness of the inner shell is usually 4 in and the material used is concrete, the former having greater resistance to very high tem-

einforcement usually consists of ½-in rods set from 1 to 2 ft on placed vertically.

well to extend the lining up at least one-half the height in order racking.

resistance to wind-pressure is taken care of by the outer main is designed to resist also the temperature-stresses due to the differ-peratures of the interior and exterior of the chimney.

all chimneys, the area of the base of the chimney is designed to greatest pressure on the earth or foundation-bed caused by the the chimney itself and the increased vertical pressure due to the the calculations for the resistance of the structure to wind-pressures sume from 25 to 50 lb per sq ft on the projected area. The latter wever, probably exceeds any ever exerted against a structure of

riments which have been made on the effect of winds of high inst flat surfaces seem to indicate that a pressure of about 20 or 25 are actually safe theoretical maximums.

aining Steel Chimneys are largely used, especially for tall chimneys s and power-houses from 150 to 300 ft in height.

antages claimed are: Greater strength and safety; smaller space naller cost by 30 to 50% as compared with brick chimneys; avoid-ltration of air and consequent checking of the draught, common

Sizes of Foundations for S

Diam., clear, in feet
Height
Least diam. of foundation
Least depth of foundation
Height
Least diam. of foundation
Least depth of foundation

---

* These dimensions were taken fro
ing Works.

(1) HYDRAULICS

r is practically an incompressible liquid, weighing, at the average temper- 62° F., 62.355 lb to the cu ft and 8.335 lb to the gallon. These figures lightly with changes in temperature and atmospheric pressure, and a riation for the same temperature will be found in different works.

ure of Water. The pressure of still water in pounds per square inch he sides of any pipe or vessel of any shape whatever is due alone to the height of the surface of the water above the point considered pressed d is equal to 0.433 lb per sq in for every foot of head at 62° F. The ssure per square inch is equal in all directions. To find the total pres- uiet water against and perpendicular to any surface, whether vertical, l, or inclined at any angle, whether it be flat or curved, multiply the area in square feet of the surface pressed, the vertical depth of its gravity below the surface of the water, and the constant 62.4. The will be the required pressure in pounds. This may be expressed by as follows:

$$P = 62.4\,AD$$

the pressure in pounds of quiescent water on the surface considered;
the area pressed upon in square feet; and
the vertical depth in feet of center of gravity of surface considered.

e A. Pressure in Pounds per Square Inch for Different Heads of Water in Feet

0	1	2	3	4	5	6	7	8	9

Flow of Water in Pipes. Owing to the many practi
ditions which affect the flow of water in pipes, such as t
pipe, number and character of the joints, bends and valv
nothing of the size and length of the pipe, all formulas
discharge of water in and through pipes can only be consid
The following formulas and data are taken largely from
Company's Book of Standards, 1902 edition. They ag
similar tables by Kent and Trautwine, both of whom devo
subject. The QUANTITY OF WATER passing through a give
the sectional area of the pipe or outlet and the mean VEL
depends primarily upon the PRESSURE OF HEAD, and is
FRICTION, which again varies with the smoothness of th
and length of the pipe, and whatever obstructions there
The HEAD is the vertical distance from the surface of the
to the center of gravity of the lower end of the pipe when
the air, or to the level surface of the lower reservoir when
water. When the pressure is produced by mechanical me
in feet may be readily determined by the following table:

Table B.* **For Converting Pressure in Pounds per Squa**
Water in Feet

Pressure	0	1	2	3	4	5	6
0	......	2.309	4.619	6.928	9.238	11.547	13.857
10	23.0947	25.404	27.714	30.023	32.333	34.642	36.952
20	46.1894	48.499	50.808	53.118	55.427	57.737	60.046
30	69.2841	71.594	73.903	76.213	78.522	80.831	83.141
40	92.3788	94.688	96.998	99.307	101.62	103.93	106.24
50	115.4735	117.78	120.09	122.40	124.71	126.02	129.33
60	138.5682	140.88	143.19	145.50	147.81	150.12	152.42
70	161.6629	163.97	166.28	168.59	170.90	173.21	175.52
80	184.7576	187.07	189.38	191.69	194.00	196.31	198.61
90	207.8523	210.16	212.47	214.78	217.09	219.40	221.71

---

* Tables A and B are exact for water at 62° F. and for atmo
lb per sq in.

To find the velocity of water discharged from a
four times its diameter, knowing the head, length and ins
following formula:

$$v = m\sqrt{\frac{hd}{L + 54\,d}}$$

in which

47	52	54	56	60	64	67
50	54	56	58	62	66	70
51	55	58	60	64	67	70

ie head, $h = 50$ ft; the length, $L = 5\,280$ ft and the diam-
the velocity and quantity of discharge.
values in the foregoing formula, we get

$$\frac{\times h}{54\,d} = \sqrt{\frac{2 \times 50}{5\,280 + 108}} = \sqrt{\frac{100}{5\,388}} = 0.136$$

$\sqrt{\frac{hd}{L + 54\,d}}$ find 0.10, which is the value nearest to 0.136,
line until column headed 2 is reached; then read 62 as
t m.

6 = 8.432 ft per sec, the velocity required.

harge in cubic feet per second, multiply this velocity by
f pipe in square feet.

× 8.432 = 26.49 cu ft per sec.

3 gal in a cubic foot, the discharge in gallons per second =

is only an approximation, since the flow is modified by
tions, etc.

in feet necessary to give a stated discharge in cubic feet,

$$h = \frac{0.000704\,Q^2\,(L + 54\,d)}{d^5}$$

The following formula is simpler and can be used w
L is so small as to be negligible:

$$h = \frac{0.000704\,Q^2 \times L}{d^5}$$

If the pipe instead of being straight has easy curves
than five diameters of the pipe) either horizontal or ve
not be materially diminished so long as the total heads
of pipe remain the same, but it is advisable to make t
than five diameters as can conveniently be done.

To find the diameter of a pipe of given length to del
water under a given head use the following,

$$d = 0.234\sqrt[5]{\frac{Q^2L}{h}}$$

in which

d = diameter of pipe in feet;
Q = cubic feet per second delivered;
L = length of line in feet;
h = head in feet.

Example. Given the head, $h = 700$ ft; the length
the quantity to be delivered, $Q = 4$ cu ft per sec; requi
necessary.

Substituting these values in the foregoing formula, w

$$d = 0.234\sqrt[5]{\frac{16 \times 3\,000}{700}} = 0.234\sqrt[5]{68.57} = 0.5$$

To find the diameter of pipe required to deliver a
with a given head.

Rule. (1) Reduce the head to feet per 100 ft; (2) f
find the discharge for the head thus obtained through
(3) divide the required discharge by that obtained fro
quotient in the column of Table D, page 1300, headed
and opposite it, in the adjoining columns of the tab
quired diameter.

Note. The use of Tables C and D gives results suff
less than 700 diameters in length.

Example. If the head of water from a reservoir to th

Velocities and Discharges Through a Straight, Smooth Pipe One in Diameter and One Mile, or 5 280 Diameters, in Length

et	Head in feet per mile	Velocity in feet per sec	Discharge in cubic feet per sec	Discharge in cubic feet per 24 hours
	3	1.13	0.8914	76 982
	4	1.31	1.028	88 862
	5	1.47	1.150	99 403
	6	1.61	1.264	109 209
	7	1.74	1.366	118 022
	8	1.86	1.455	125 740
	9	1.96	1.539	132 969
	10	2.08	1.633	141 145
	12	2.27	1.782	153 964
	14	2.45	1.924	166 233
	16	2.62	2.057	177 724
	18	2.78	2.183	188 611
	20	2.93	2.301	198 806
	25	3.28	2.572	222 156
	30	3.59	2.819	243 604
	35	3.88	3.047	263 260
	40	4.15	3.267	282 288
	45	4.40	3.451	298 209
	50	4.64	3.638	314 352
	60	5.08	3.989	344 649
	70	5.49	4.311	372 470
	80	5.85	4.602	397 613
	90	6.23	4.900	423 435
	100	6.56	5.144	444 312
	110	6.87	5.395	466 128
	120	7.18	5.639	487 209
	130	7.47	5.866	506 822
	140	7.76	6.094	526 521
	150	8.05	6.322	546 048
	160	8.30	6.534	564 576
	170	8.55	6.715	580 176
	180	8.80	6.903	596 418
	190	9.04	7.100	613 440
	200	9.28	7.276	628 704
	225	9.84	7.696	664 848
	250	10.4	8.168	705 728
	275	10.8	8.482	732 844
	300	11.3	8.914	769 824
	350	12.3	9.621	831 168
	400	13.1	10.28	888 624
	450	13.9	10.91	943 056
	500	14.7	11.50	994 032
	550	15.4	12.09	1 044 576
	600	16.1	12.64	1 092 096
	650	16.7	13.11	1 132 704
	700	17.4	13.66	1 180 224
	750	18.0	14.13	1 220 832
	800	18.6	14.55	1 257 408
	850	19.1	15.00	1 296 000
	900	19.6	15.39	1 329 696
	950	20.3	15.94	1 377 216
	1 000	20.8	16.33	1 411 456
	1 200	22.7	17.82	1 539 648
	1 400	24.5	19.24	1 662 336
	1 600	26.2	20.57	1 777 248
	1 800	27.8	21.83	1 886 112
	2 000	29.3	23.01	1 988 064
	2 500	32.8	25.72	2 221 560
	3 000	35.9	28.19	2 436 040

Table D. Diameters of Pipes and Ratio of Discharge

Diameter of pipe, in	Diameter of pipe, ft	Ratio of discharge to that through a 1-ft pipe with the same head per mile	Diameter of pipe, in	Diameter of pipe, ft
1	0.0833	0.0020	12½	1.042
1½	0.1250	0.0055	13	1.083
2	0.1667	0.0113	14	1.167
2½	0.2083	0.0198	15	1.250
3	0.2500	0.0310	16	1.333
3½	0.2917	0.0458	17	1.417
4	0.3333	0.0643	18	1.5
4½	0.3750	0.0857	19	1.583
5	0.4167	0.1119	20	1.667
5½	0.4583	0.1422	21	1.75
6	0.5	0.1767	22	1.833
6½	0.5417	0.2159	23	1.917
7	0.5833	0.2600	24	2
7½	0.6250	0.3090	24⅝	2.052
8	0.6667	0.3631	26	2.167
8½	0.7083	0.4220	28	2.333
9	0.75	0.4871	30	2.5
9½	0.7917	0.5575	30¼	2.521
10	0.8333	0.6337	32	2.667
10½	0.8750	0.7157	34	2.833
11	0.9167	0.8044	36	3
11½	0.9583	0.8987	38	3.167
12	1	1	40	3.333

This table shows, also, the relative discharging capacities of long pip
one 12-in pipe is equal to two 9-in pipes, to nearly six 6-in pipes, o
three 3-in pipes.

Table E. Flow of Water in House Service-Pipes

Thomson Meter Company

To find the discharge in gallons, multiply by 7.47

n of ge	Pressure in main, lb per sq in	Discharge in cubic feet per minute from the pipe								
		Nominal diameters of iron or lead service-pipe in inches								
		½	⅝	¾	1	1½	2	3	4	6
f e- no re	30	1.10	1.92	3.01	6.13	16.58	33.34	88.16	173.85	444.63
	40	1.27	2.22	3.48	7.08	19.14	38.50	101.80	200.75	513.42
	50	1.42	2.48	3.89	7.92	21.40	43.04	113.82	224.44	574.02
	60	1.56	2.71	4.26	8.67	23.44	47.15	124.68	245.87	628.81
	75	1.74	3.03	4.77	9.70	26.21	52.71	139.39	274.89	703.03
	100	2.01	3.50	5.50	11.20	30.27	60.87	160.96	317.41	811.79
	130	2.29	3.99	6.28	12.77	34.51	69.40	183.52	361.91	925.58
of c- no re	30	0.66	1.16	1.84	3.78	10.40	21.30	58.19	118.13	317.23
	40	0.77	1.34	2.12	4.36	12.01	24.59	67.19	136.41	366.30
	50	0.86	1.50	2.37	4.88	13.43	27.50	75.13	152.51	409.54
	60	0.94	1.65	2.60	5.34	14.71	30.12	82.30	167.06	448.63
	75	1.05	1.84	2.91	5.97	16.45	33.68	92.01	186.78	501.58
	100	1.22	2.13	3.36	6.90	18.99	38.89	106.24	215.68	579.18
	130	1.39	2.42	3.83	7.86	21.66	44.34	121.14	245.91	660.36
of e- nd er- ise	30	0.55	0.96	1.52	3.11	8.57	17.55	47.90	97.17	260.56
	40	0.66	1.15	1.81	3.72	10.24	20.95	57.20	116.01	311.09
	50	0.75	1.31	2.06	4.24	11.67	23.87	65.18	132.20	354.49
	60	0.83	1.45	2.29	4.70	12.94	26.48	72.28	146.61	393.13
	75	0.94	1.64	2.59	5.32	14.64	29.96	81.79	165.90	444.85
	100	1.10	1.92	3.02	6.21	17.10	35.00	95.55	193.82	519.72
	130	1.26	2.20	3.48	7.14	19.66	40.23	109.82	222.75	597.31
of e- nd er- ise	30	0.44	0.77	1.22	2.50	6.80	14.11	38.63	78.54	211.54
	40	0.55	0.97	1.53	3.15	8.68	17.79	48.68	98.98	266.59
	50	0.65	1.14	1.79	3.69	10.16	20.82	56.98	115.87	312.08
	60	0.73	1.28	2.02	4.15	11.45	23.47	64.22	130.59	351.73
	75	0.84	1.47	2.32	4.77	13.15	26.95	73.76	149.99	403.98
	100	1.00	1.74	2.75	5.65	15.58	31.93	87.38	177.67	478.55
	130	1.15	2.02	3.19	6.55	18.07	37.02	101.33	206.04	554.96

may also be used when the pressure is in feet-head of water by reducing
in feet to pounds per square inch by Table A. Thus, if we wish the
per minute through a ¾-in pipe 100 ft long with a head of 70 ft, we
Table A that a head of 70 ft corresponds to a pressure of 30 lb per sq
om Table E we find the discharge through a ¾-in pipe 100 ft long with
of 30 lb to be 1.84 cu ft per minute.

Table F. Friction of Water in Pipes Based on Ellis and Ho Experiments

The following table gives the friction-loss in pounds-pressure per s for EACH 100 ft of length in clean iron pipes of different sizes, dischar quantities of water per minute. This friction-loss is greatly increase or irregularities in the pipe.

To find the friction-head in feet, multiply by 2.3

Gallons per minute	Sizes of pipes, inside diameter						
	¾ in	1 in	1¼ in	1½ in	2 in	2½ in	3 in
5	3.3	0.84	0.31	0.12	......	......	......
10	13.0	3.16	1.05	0.47	0.12	......	......
15	28.7	6.98	2.38	0.97	0.26	......	......
20	50.4	12.3	4.07	1.66	0.42	......	......
25	78.8	19.0	6.40	2.62	0.64	0.21	0.1
30	......	27.5	9.15	3.75	0.91	......	......
35	......	37.0	12.4	5.05	1.22	......	......
40	......	48.0	16.1	*6.52	1.60	......	0.2
45	......	......	20.2	8.15	2.02	......	......
50	......	......	24.9	10.0	2.44	0.81	0.3
75	......	......	56.1	22.4	5.32	1.80	0.7
100	......	......	......	39.0	9.46	3.20	1.3
125	......	......	......	......	14.9	4.89	1.9
150	......	......	......	......	21.2	7.00	2.8
175	......	......	......	......	28.1	9.46	3.8
200	......	......	......	......	37.5	12.47	5.0
250	......	......	......	......	......	19.66	7.7
300	......	......	......	......	......	28.06	11.
350	......	......	......	......	......	......	15.
400	......	......	......	......	......	......	19.5
450	......	......	......	......	......	......	25.0
500	......	......	......	......	......	......	30.8
600	......	......	......	......	......	......	......
700	......	......	......	......	......	......	......

Water-Pipe is usually tested to 300 lb pressure per square inch befo and a hammer-test should be made while the pipe is under pressure. length for each section of cast-iron water-pipe is from 12 ft 4 in to 1 pending upon the depth of the socket, each length making approxin of pipe when laid. Pipes from 2 to 4 in diameter are sometimes m 9-ft lengths.

Pressures and Equivalent Heads of Water for Cast-Iron Pipes of Different Sizes and Thicknesses

Calculated by F. H. Lewis from Fanning's Formula

Size of pipe, in											
4		6		8		10		12		14	
Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft
112	258	49	112	18	42	....	....	....	....	....	....
224	516	124	280	74	171	44	101	24	55	....	....
336	774	199	458	130	300	89	205	62	143	42	97
...	...	274	631	186	429	132	304	99	228	74	170
...	...	...	...	...	...	177	408	137	316	106	244
...	...	...	...	...	...	224	516	174	401	138	316
...	...	...	...	...	...	....	....	212	488	170	392
...	...	...	...	...	...	....	....	249	574	202	465
...	...	...	...	...	...	....	....	....	....	234	538
...	...	...	...	....	...	....	....	....	....	266	612
16		18		20		24		30		36	
56	129	41	95	...	...	....	....	....	....	....	....
84	194	66	152	51	118	30	69	....	....	....	....
112	258	91	210	74	170	49	113	24	55	....	....
140	323	116	267	96	221	68	157	39	90	....	....
168	387	141	325	119	274	86	198	54	124	32	74
196	452	166	382	141	325	105	242	69	159	44	101
224	516	191	440	164	378	124	286	84	194	57	131
...	...	216	497	209	481	161	371	114	263	82	189
...	...	...	...	256	589	199	458	144	332	107	247
...	...	...	...	...	...	237	546	174	401	132	304
...	...	...	...	...	...	....	....	204	470	157	362
...	...	...	...	...	...	....	....	234	538	182	419
...	...	...	...	...	...	....	....	....	....	207	477

Weights of Lead and Gaskets for Pipe-Joints

Dennis Long & Company

ter e,	Lead, lb	Gasket, lb	Diameter of pipe, in	Lead, lb	Gasket, lb
	2.5	0.125	12	15	0.250
	3.5	0.170	14	18	0.375
	4.5	0.170	16	22	0.500
	6.5	0.200	18	26	0.500
	9.0	0.200	20	33	0.625
	13.0	0.250	....	....	....

Weights, per Foot, of Cast-Iron Pipes in General Use, Including Sock and Spigot-ends

Dennis Long & Company, Inc., Louisville, Ky.

Diameter, in	Thickness, in	Weight per ft, lb	Diameter, in	Thickness, in	Weight per ft, lb	Diameter, in	Thickness, in
3	3/8	12½	16	3/4	129	30	2
	7/16	15		7/8	152	36	3/4
	1/2	18		1	175		1
	9/16	20½	18	5/8	120		1 1/8
	5/8	23		3/4	146		1 1/4
4	3/8	17		7/8	171		1 3/8
	7/16	20		1	197		1 1/2
	1/2	23½		1 1/8	223		1 5/8
	9/16	26¾		1 1/4	249		1 3/4
	5/8	30	20	11/16	148		1 7/8
6	7/16+	30		3/4	161		2
	1/2	34		7/8	190	42	1
	9/16	38¼		1	216		1 1/8
	5/8	42½		1 1/8	247		1 1/4
	3/4	52		1 1/4	276		1 3/8
8	7/16	40		1 3/8	305		1 1/2
	1/2	43½		1 1/2	334		1 5/8
	9/16	49¾	24	3/4	191		1 3/4
	5/8	56		7/8	225		1 7/8
	3/4	68		1	258		2
10	7/16	50		1 1/8	293	48	1 1/8
	1/2	54		1 1/4	327		1 1/4
	9/16	60		1 3/8	361		1 3/8
	5/8	68		1 1/2	395		1 1/2
	3/4	82		1 5/8	430		1 5/8
12	1/2	70		1 3/4	465		1 3/4
	9/16	76	30	13/16	258		1 7/8
	5/8	82		7/8	278		2
	3/4	99		1	319	60	1 1/4
	7/8	117		1 1/8	360		1 3/8
14	9/16	85		1 1/4	405		1 1/2
	5/8	94		1 3/8	448		1 5/8
	3/4	113		1 1/2	489		1 3/4
	7/8	137		1 5/8	532		1 7/8
16	9/16	100		1 3/4	575		2
	5/8	108		1 7/8	619		2 1/4

There is no standard weight of pipe for any given pressure.

Private Water-Supply. Pumps

Private Water-Supplies. The architect is frequently required to a water-supply for isolated buildings, and even in cities it is becomin common for manufacturing establishments and large buildings to hav own water-supply; so that some knowledge of the various methods plying water is requisite. Power-pumps are of so many kinds and cate in construction that no attempt will be made to describe them.

The Hydraulic Ram. Where a small stream of water having a fall or more flows near the premises, an hydraulic ram may be used to g

Table of Water Required for Rife Rams

Number	Dimensions						Size of drive-pipe, in	Size of delivery-pipe, in	Gallons per minute required to operate engine, gal	Least no. of feet of fall recommended, ft
	Height, ft	in	Length, ft	in	Width, ft	in				
10	2	1	3	2	1	8	1¼	¾	3 to 6	3
15	2	1	3	4	1	8	1½	¾	5 to 12	3
*20	2	3	3	8	1	9	2	1	10 to 18	2
25	2	3	3	9	1	9	2½	1	11 to 24	2
30	2	7	3	10	1	10	3	1¼	15 to 35	2
40	3	3	4	4	2	0	4	2	30 to 75	2
80	7	4	8	4	2	8	8	4	150 to 350	2
*120	......		......		......		12	5	375 to 750	2
†120	8	9	9	6	3	8	12 (two)	6	750 to 1 500	2

* Single. † Duplex.

Table of Capacities of Rife Rams

Power-head or fall in ft	Height or head in feet the water is to be delivered														
	4	10	15	20	30	40	*50	60	70	80	90	100	120	140	160
2	540	192	128	96	64	43	29	24	....	....	....	....	....	....	....
3	...	301	192	144	96	72	58	43	37	27	24	....	....	....	....
4	...	432	256	192	128	96	77	64	55	43	38	29	24	....	....
5	...	540	345	240	160	120	96	80	69	60	53	43	30	26	....
6	...	...	432	302	192	144	115	96	82	72	64	57	43	31	27
7	...	...	505	378	235	168	134	112	96	84	75	67	50	36	31
8	...	...	...	432	270	192	154	128	110	96	86	77	64	55	43
9	...	...	...	485	300	216	173	144	124	108	96	86	72	62	54
*10	...	...	...	540	360	252	*192	160	137	120	107	96	80	68	60
12	...	...	...	...	430	301	230	192	165	144	128	115	96	82	72
14	...	...	...	...	505	353	270	224	192	168	150	135	112	96	84
16						432	323	257	220	192	171	154	128	110	96
18						486	390	303	247	216	192	173	144	124	108
20						540	430	336	288	240	214	192	160	137	120
22						....	475	370	303	264	235	212	176	151	132
24						....	520	405	346	288	256	230	192	164	144
26						....	....	470	375	328	278	250	208	178	156
28						....	....	505	430	354	300	260	224	192	168
30						....	....	540	465	405	336	288	240	206	180

Example

With a supply of 1 400 gal per min, 10-ft fall, 50-ft elevation, No. 120 engine will deliver 268 800 gal per day. 1 400 × 192 = 268 800

* Multiply factor opposite POWER-HEAD and under PUMPING-HEAD by the gallons per minute USED by the engine; the result will be the number of gall ERED per day.

The efficiency developed is governed by the ratio of fall to pumping-head.

The efficiency of rife rams is based on....
75% for a ratio of 1 to
70% for a ratio of 1 to 3
66⅔% for a ratio up to 1 to
60% for a ratio up to 1 to
50% for a ratio up to 1

ed by hand, or by a crank-rod attached to a pumping-jack,
. With a single-acting pump the plunger is raised and
every revolution of the driving-wheel, the principle of oper-
ne as in an ordinary hand suction-pump. Fig. 2 shows a
t for operating a working-head by belt-power. This is
ll power working-head. A DEEP-WELL PUMP (Fig. 2) differs
IP in that it will raise water from any depth, whereas a suc-
ice will raise water only about 20 ft. A suction-pump may
oint in relation to the well, and will draw the water any
al distance. The deep-well pump, on the other hand, must
the well, but it will then deliver the water at any desired
t of water pumped in a minute by any single-acting pump
e diameter of the suction-cylinder, the length of stroke, and
kes per minute. The table following gives the capacity per
of different diameters, and for strokes of different lengths.
y per minute, multiply the values given in the table by the
nute. The usual speed of single-acting working-heads and
m 25 to 30 revolutions per minute. Cylinders over 2¾ in in
ve a substantial iron working-head.

apacity of Single-Acting Pumps of Given Diameter and Length of Stroke

Length of stroke in inches							
	10	12	14	16	18	20	24
Capacity per stroke in gallons							
425	0.0531	0.0637	0.0743	0.0848	0.0955	0.1062	0.1274
513	0.0642	0.0770	0.0890	0.1027	0.1156	0.1280	0.1541
612	0.0765	0.0918	0.1071	0.1224	0.1377	0.1530	0.1836
833	0.1041	0.1249	0.1457	0.1666	0.1874	0.2082	0.2499
088	0.1360	0.1632	0.1904	0.2176	0.2448	0.2720	0.3264
377	0.1721	0.2063	0.2410	0.2754	0.3096	0.3442	0.4128
700	0.2125	0.2550	0.2975	0.3400	0.3825	0.4250	0.5100
057	0.2571	0.3085	0.3598	0.4114	0.4626	0.5142	0.6170
448	0.3060	0.3672	0.4284	0.4896	0.5508	0.6120	0.7344
872	0.3594	0.4312	0.5030	0.5748	0.6466	0.7182	0.8624
332	0.4165	0.4998	0.5831	0.6664	0.7497	0.8330	0.9996
824	0.4780	0.5736	0.6692	0.7648	0.8605	0.9561	1.1470
352	0.5440	0.6528	0.7616	0.8704	0.9792	1.0880	1.3056
912	0.6141	0.7368	0.8596	0.9824	1.1050	1.2280	1.4730
508	0.6885	0.8262	0.9639	1.1016	1.2393	1.3770	1.6524
136	0.7671	0.9204	1.0730	1.2270	1.3800	1.5340	1.8400

s. These are very extensively used for pumping water for
they are absolutely *safe, require little attention,* and have
r gauges to get out of order. They are also adapted to al-
-l, such as coal, coke, wood, gas, or kerosene oil. They
a shallow or a deep well, but are best adapted to wells

in which the surface of the water is within 20 ft of the top of the
best known hot-air engines are the Rider-Ericsson, which have bee
ful operation for many years. These engines have capacities rang
to 3 500 gal per hour and will deliver water from 50 to 350 f
surface of water in the well, although the higher the water is ra
will be the quantity delivered. The cost of these engines, wit
tached, varies from $110 for the smallest size, having a capacity o
hour raised 50 ft, to $540 for the largest size, having a capacity of
hour raised 50 ft. The smaller size requires about 1 quart of ker
of anthracite coal per hour. Hot-air engines should be placed
source of supply, and when the latter is a deep well the eng
placed so that the pump-rod will be in a vertical line above th
the well, the operation of pumping being the same as that of
single-acting deep-well pump. It is not practicable to DRAW wat
from 20 to 25 ft, in height, with any form of suction-pump, be
difficulty of keeping the pipe, valve and fittings absolutely ai
further information, see the catalogue of the Rider-Ericsson Engi

Action of Wind and Capacities of Pumping Windmill

Velocity per hour in miles	Pressure* per square foot in pounds	Description of wind	Action of wind an
3	0.045	Just perceptible........	Windmills will no
5	0.125	Pleasant wind..........	Might start if ligh
8	0.33	Fresh breeze...........	Will start pumping
10	0.5	Average wind..........	Pumps nicely if pr
15	1.125	Good working wind...	Does excellent wo
20	2	Strong wind...........	Gives best service
25	3.125	Very strong wind......	Maximum results
30	4.5	Gale..................	Should be furled c
40	8	Storm.................	Well-constructed towers safe if pro
50	12.5	Severe storm..........	
60	18	Violent storm.........	Buildings, trees, e injured
80	32	Hurricane.............	Buildings, trees, be injured
100	50	Tornado...............	Ruin

From the above table it will be seen that the only available winds are
with a velocity of from 8 to 25 miles per hour, and that a 15-mile wind
to the best advantage. It is therefore advisable to LOAD a windmill for a
It then starts pumping in an 8-mile wind, does excellent work in a 15-
reaches the maximum results in a 25-mile wind.

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* The pressures per square foot in pounds will vary slightly from th
according to the formula which is used to obtain such pressures. See,
XXVII, pages 1052-3 and also page 1637.

Windmills. In the country and on large suburban estates, w
extensively used for pumping water. Aside from the noise of o
only objection to the windmill, where it can be used, is the irreg
supply, but with a large storage-tank this is not a serious objectio
for domestic purposes only. Professor Thurston says, regard
"In estimating the capacity, a working-day of eight hours is

ine, when used for pumping, may actually do its work twenty-four hours a or days, weeks, and even months together, whenever the wind is stiff enough rn it. It costs for work done only one-half or one-third as much as steam, ir, or gas-engines of similar power." The action of wind of different ities, the pressure per square foot of sail-surface and its relation to the ing capacity of pumps can be found in the following table, compiled by anks, Morse & Company.

e windmill operates the plunger in the well, the process of pumping being ame as that of the single-acting pumps described above. The following of capacity was prepared by Alfred R. Wolff, and is sufficiently accurate ll practical purposes:

Capacity of the Windmill

sig- ion mill eel, t	Velocity of wind in miles per hour	Revolutions of wheel per minute	Gallons of water raised per minute to an elevation of						Equivalent actual useful h.p. developed
			25 ft	50 ft	75 ft	100 ft	150 ft	200 ft	
½	16	40 to 50	6.192	3.016	....	....	....	....	0.04
	16	35 to 40	19.179	9.563	6.638	4.750	....	....	0.12
	16	30 to 35	33.941	17.952	11.851	8.435	5.680	....	0.21
	16	28 to 35	45.139	22.569	15.304	11.246	7.807	4.998	0.28
	16	25 to 30	64.600	31.654	19.542	16.150	9.771	8.075	0.41
	16	22 to 25	97.682	52.165	32.513	24.421	17.485	12.211	0.61
	16	20 to 22	124.950	63.750	40.800	31.248	19.284	15.938	0.78
	16	16 to 18	212.381	106.964	71.604	49.725	37.349	26.741	1.34

e horse-power of windmills of the best construction is proportional to the es of their diameters and inversely as their velocities; for example, a 10-ft n a 16-mile breeze will develop 0.15 horse-power at 65 revolutions per min- and with the same breeze:

a 20-ft mill, at 40 revolutions per minute, 1 horse-power;
a 25-ft mill, at 35 revolutions per minute, 1¾ horse-power;
a 30-ft mill, at 28 revolutions per minute, 3½ horse-power.

e wheels of very few windmills are larger than 25 ft in diameter. There o pumps which will enable the user of a windmill to utilize the increased r obtained from winds of high velocity, so that in practice the amount of pumped by windmills in high winds is but little more than is pumped by me mills in winds having velocities of from 12 to 18 miles per hour. For eason it is customary to regulate windmills to govern at about 25 miles an

Theoretically the increase in power from increased velocity of wind is equal e square of its proportional velocity; as, for example, the 25-ft mill rated for a 16-mile wind will, with a 32-mile wind, have its horse-power increased × 1¾ = 7 horse-power. A windmill "will run and produce work in an 8- breeze." Windmills have also been used for the generating and storage ctricity for small lighting-plants.*

-Lift Process. Compressed air is now being used to an increasing extent ising water from artesian wells. *The process in general consists of sub-* ing a dis*charge-pipe in a closed well, with a smaller pipe inside* delivering

---

* *See Kent's Mechanical Engineers' Pocket-Book.*

compressed air into it at the botto
pansive force lifts a column of min
open tank, to permit of the escape o
conveyed by gravity into a series o
different parts of a building, the
pressor and power for driving it.
loss of energy in the air-lift. The
its general conditions and the quant
are alike, and consequently the m
one would be unsuited to another.
piping any particular well may be
Company.

Advantages of the Air-Lift Proce
may be obtained from a given diam
because there are no valves, cylind
water. One air-compressor operat
distance apart so as not to affect
engine-room to look after or wear o
Sand or gravel does no harm. Th
method, including fuel, labor, oil,
foundations, pipes, real estate, er
ciation, runs from 2½ cts down to
height of lift, and other local cond
or small size, it is usually under
sively used in ice-works, breweries,
etc., and a great variety of indu
quite a number of the smaller citi
are at work having a total capacit
three 8-in artesian wells.†

Pneumatic Water-Supply Sys
water to buildings is used extensive
public water-supplies. With the
vated tank, a closed water-tight ta
be located at any level, for the wat
air confined in the top of the tank.
in the ground below the frost-line, a
water will have an almost uniform
protected from possible contamina
agencies, while the tank takes up
weight upon the attic-floor of a bu
The principle of operation is this:
then, water is pumped into a clos
within, and the more water pumpe
The elasticity of the air, then, will f
opened, and the water will continue t
sure in the tank. In practice the ai
tank, and in a short time become ex
This is accomplished by injecting a
of the pump, by means of a SNIFTER
connections to the tank are taken f
which would occur if the connectio

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* Ingersoll-Sergeant

wer Required to Raise Water to Different Heights

iciples. The power required to raise a certain quantity of water to
varies directly with the quantity to be raised, and also with the
tance, it requires twice as much power to raise 200 gal per minute
oes to raise 100 gal to the same height and in the same time; and to
ft high requires twice as much power as it does to raise 100 gal 10
the theoretical horse-power necessary to elevate water to a given
the number of gallons per minute by 8.335, the weight of 1 gal,
by the total number of feet the water is raised, that is, from the
vater to the highest point to which the water is raised, and the
power in foot-pounds; divide by 33 000, and the quotient is
. To the theoretical power a liberal allowance must be made
ncy of the pump. For a cylinder-pump add from 75 to 100%,
eight to which the water is to be raised add the friction-loss in
ble F, page 1302, when the discharge is to be piped any distance.

nd the theoretical horse-power required to raise 100 gal per
igh, through a 3-in pipe, 200 ft long.

om Table F, the friction-head for 100 gal per min in a 3-in pipe,
31 × 2.3 or 3 ft. For 200 ft it will be 6 ft, which, added to 120,
the height. Then theoretical horse-power = 100 × 8.35 × 126/
. The actual horse-power required will probably vary from
ng to the efficiency of the pump. The mistake of using too
ge-pipe can easily be seen from Table F. For instance, if
to force 100 gal per minute through 100 ft of 2-in pipe,
re would be equivalent to raising the water 22 ft high. The
be correspondingly increased. Right-angle turns are to be
friction is very materially increased, being practically equal to
ft of straight pipe.

Table of Effective Fire-Streams

2½-in ordinary best-quality rubber-lined hose between nozzle and hydrant or pump

th nozzle	¾ in					⅞ in				
rant, lb.	32	54	65	75	86	34	57	69	80	91
zle, lb.	30	50	60	70	80	30	50	60	70	80
ft.	48	67	72	76	79	49	71	77	81	85
ance, ft.	37	50	54	68	62	42	55	61	66	70
per min.	90	116	127	137	147	123	159	174	188	201

th nozzle	1 in					1⅛ in				
rant, lb.	37	62	75	87	100	42	70	84	98	112
zle, lb.	30	50	60	70	80	30	50	60	70	80
ft.	51	73	79	85	89	52	75	83	88	92
ance, ft.	47	61	67	72	76	50	66	72	77	81
per min.	161	208	228	246	263	206	266	291	314	336

. The following is an extract *from a paper read by* John
a meeting of the New England Waterworks Association,
xperiments and Practical Tables Relating to Fire-Streams,

"When unlined linen hose is
60%, increasing with the pres
short lengths. Mill-hose is
but ordinarily the best qualit
mill-hose, having less friction
smooth nozzle and actually
instance, the ⅞-in ring-nozz
¾-in smooth nozzle, and a 1-
Two hundred and fifty gallo
80-lb pressure at the hydran
for very high buildings or len

Notes on the Const

Material should be either
pine, free from imperfection
Gulf cypress makes the most

Staves and Bottom of tar
made of 2½-in, dressed to ab
16 ft diameter or 16 ft deep
stock should be used. For
should be connected about
dowel to hold them in positi
dressed on four sides, and th
not over 3 ft apart for ⅝-in

Taper. The batter to eac
½ in per ft.

Hoops should be of ROUND
iron is preferable because it d
welds in any of the hoops.
lugs should be used to make t
the several pieces constituting
shipment. Hoops for fire-tan
in no hoop will exceed 12 500
of thread. For general purp
On acccount of the swelling
may be subjected to a stress
additional hoops, therefore, s
eter, one hoop of the size use
opposite the croze and not c
For tanks 20 ft or more in
Hoops with UPSET ends mu
within 2 in of the top of stave
as possible. Hoops should
line. No hoop should be les
of mill-scale and rust and pai
before erecting.

Note. The strength of a
are specified because they d

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* These notes have been co
Department of the Factory M
most excellent pamphlet.

ve the same weakening effect as on a flat hoop, and round hoops are not o burst when the tank swells, as they will sink into the wood.

cing of Hoops. The hoops should be spaced so that each one will he same stress per square inch, and no space should be greater than 21 in. et this requirement the hoops must be spaced quite close together at ttom, the space between them gradually increasing towards the top.

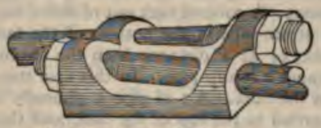


Fig. 4. Lug for Tank-hoops

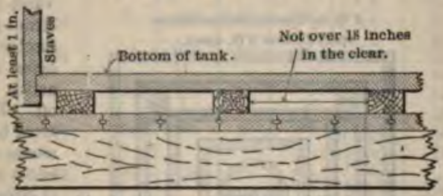


Fig. 5. Support for Bottom of Tank

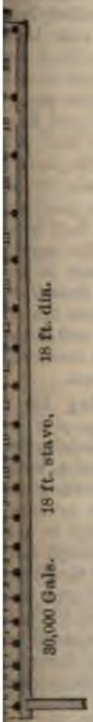


Diagram hoop-spacing anks

Fig. 3 shows the proper spacing of hoops for a tank 18 ft in diameter, with 18-ft staves. The spacing for seven other sizes of tanks is given in the pamphlet referred to. It may be computed by the following formula:

$$\text{Spacing of hoops in inches} = \frac{\text{strength}}{2.6 \times \text{diameter in feet} \times H}$$

For strength of a ¾-in rod use 3 750; of a ⅞-in rod, 5 250; of a 1-in rod, 6 875; and of a 1⅛-in rod, 8 625.

H is the distance from surface of the water to center of hoop in feet.

Example. How far apart should 1-in hoops be placed, at 15 ft 2 in from top of tank, on a tank 20 ft diameter?

Solution. $$\text{Spacing} = \frac{6\,875}{2.6 \times 20 \times 15} = 8\tfrac{3}{4} \text{ in}$$

Lugs should be as strong as the hoops. A lug similar to Fig. 4 is simple and fulfils the requirement for strength. Malleable lugs are required.

port. The weight of the tank should be supported entirely from its s; and in no event should any weight come on the bottom of the staves. lanks upon which the tank-bottom rests should cover at least one-fifth ea of the bottom, should be not over 18 in apart, and of such thickness he bottom of the staves will be at least 1 in from the floor *(see Fig. 5)*.

Discharge-Pipe *should preferably leave the bottom of the tank at its and extend up inside of the tank 4 in, to allow the sediment to collect ottom of the tank.*

The Overflow-Pipe shoul
discharging either through si
is much to be preferred to
order.

Heating. Tanks of mode
prevent freezing. When a ta
best method is to keep the ro
the boiler-room. A covered t
placing the steam-pipe in th
a high trestle, or at a dista
cable to arrange a return-pip
the water in the tank. A 1-i
should be carried to the top
wards, so that its outlet is

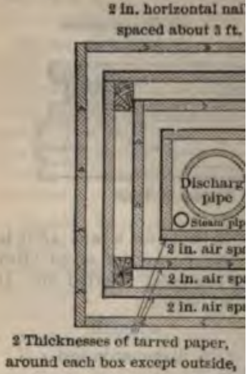


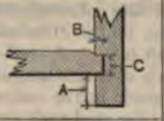
Fig. 6. Method of Fros

shown in Fig. 6. If steam i
inside the box. In New Eng
boxing is generally used, wh
double boxing is used. The
ground below the frost-line,
the tank.

Covers. For economy in
getting into the water, all
cover is recommended consist
supported by joists which spa
conical roof. To prevent the
fastened to the top of the ta
wind particular attention sh
roof rests on the top of the st

Scuttles should be arranged
to the inside of the tank and
easy access to the top of the

Dimensions of Tanks of Standard Sizes

Size. Outside dimensions			Thickness of lumber after being machined					Hoops	
verage liameter, t	in	Length of stave, ft	Staves, in	Bottom, in	*A*, in	*B*, in	*C*, in	Number of	Size, in
3	4	12	2¼	2¼	3½	⅝	2⅛	11	¾
4	6	14	2¼	2¼	3½	⅝	2⅛	14	¾
5	6	16	2¼	2¼	3½	⅝	2⅛	5 11	¾ ⅞
7	6	16	2¾	2¾	3½	¾	2⅝	4 12	¾ ⅞
8	0	18	2¾	2¾	3½	¾	2⅝	4 16	¾ ⅞
2	0	20	2¾	2¾	3½	¾	2⅝	4 19	⅞ ⅞
4	6	24	2¾	2¾	3½	¾	2⅝	4 6 21	⅞ 1 1⅛
8	6	24	2¾	2¾	3½	¾	2⅝	5 29	1 1⅛

for Fire-Streams. The dimensions of steam-pumps for fire-pro- buildings, approved by the Board of Underwriters, can be found in ng table.

Underwriter Steam Fire-Pumps

n inches		Boiler h.p. required, A.S.M.E. standard	Size in inches							Over-all dimensions of largest pump of given capacity						Proper capacity for priming-tank
Diameter of water-plunger	Length of stroke		Suction-pipe	Discharge-pipe	Steam supply-pipe	Steam exhaust-pipe	Relief-valve	Piston-rod	Valve-rod	Length		Width		Height		
in	in	h.p.	in	in	in	in	in	in	in	ft	in	ft	in	ft	in	gal
7 7¼ 8	12 12 10	80	8	6	3	4	3	2	1	9	⅛	5	2	7	5	250
9 9¼	12 12	115	10	7	3½	4	3½	2¼	1⅛	9	5	5	2	8	0	375
10 10½	12 10	150	12	8	4	5	4	2⅜	1⅛	10	8	5	7½	8	10	500
12	16	200	14	10	5	6	5	2½	1¼	12	5	5	7	8	11	750

ities given in *last column are desirable; but in case the suction-pipe is short* *ow, a tank of not less than one-half the capacity stated may sometimes* **be**

Notes on Steel Tanks*

Steel Tanks of sizes commonly used for fire-protection cost fro
more than wooden tanks. The additional cost for large tanks
less than for small tanks. A steel tank of about 40 000-gal cap
can be erected on a steel trestle at about the same cost as a wood
a saving can be made in the cost of supports by making a hem
conical bottom to the steel tank and supporting the tank directl
of the trestle, thus saving the expense of horizontal supporting be
tank is superior to a wooden tank for the following reasons: (1
for an indefinite time if KEPT THOROUGHLY PAINTED inside and o
wooden tank will have to be replaced in from twelve to thirty
in about fifteen years; (2) it will be absolutely tight when onc
and properly cared for, whereas a wooden tank will shrink and lea
gets low; (3) it will not be at all likely to burst suddenly, if origin
designed, even if painting is neglected, for experience shows tha
will first rust through and thus show the weak condition by small
a wooden tank, if neglected, may burst its hoops suddenly and
damage. The objections to steel tanks are that: (1) They require
makers to erect them, thus adding considerable to the cost when
distance from a boiler-shop; (2) they are more difficult to protect
ing; (3) they give more trouble by SWEATING when placed in a m
they deteriorate rapidly if painting is neglected.

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* Inspection Department of the Factory Mutual Insurance Compan

a Cubic Feet and U. S. Gallons of Pipes and Cylinders of Various Diameters and One Foot in Length

1 gallon = 231 cu in. 1 cu ft = 7.4805 gal

or 1 ft in length		Diameter in inches	For 1 ft in length		Diameter in inches	For 1 ft in length	
ft, ɔ in t	U. S. gal 231 cu in		Cu ft, also area in sq ft	U. S. gal, 231 cu in		Cu ft, also area in sq ft	U. S. gal, 231 cu in
03	0.0025	6¾	0.2485	1.859	19	1.969	14.73
05	0.0040	7	0.2673	1.999	19½	2.074	15.51
08	0.0057	7¼	0.2867	2.145	20	2.182	16.32
10	0.0078	7½	0.3068	2.295	20½	2.292	17.15
14	0.0102	7¾	0.3276	2.450	21	2.405	17.99
17	0.0129	8	0.3491	2.611	21½	2.521	18.86
21	0.0159	8¼	0.3712	2.777	22	2.640	19.75
26	0.0193	8½	0.3941	2.948	22½	2.761	20.66
31	0.0230	8¾	0.4176	3.125	23	2.885	21.58
36	0.0269	9	0.4418	3.305	23½	3.012	22.53
42	0.0312	9¼	0.4667	3.491	24	3.142	23.50
48	0.0359	9½	0.4022	3.682	25	3.409	25.50
55	0.0408	9¾	0.5185	3.879	26	3.687	27.58
85	0.0638	10	0.5454	4.080	27	3.976	29.74
23	0.0918	10¼	0.5730	4.286	28	4.276	31.99
67	0.1249	10½	0.6013	4.498	29	4.587	34.31
18	0.1632	10¾	0.6303	4.715	30	4.909	36.72
76	0.2066	11	0.6600	4.937	31	5.241	39.21
41	0.2550	11¼	0.6903	5.164	32	5.585	41.78
12	0.3085	11½	0.7213	5.396	33	5.940	44.43
91	0.3672	11¾	0.7530	5.633	34	6.305	47.16
76	0.4309	12	0.7854	5.875	35	6.681	49.98
68	0.4998	12½	0.8522	6.375	36	7.069	52.88
67	0.5738	13	0.9218	6.895	37	7.467	55.86
73	0.6528	13½	0.9940	7.436	38	7.876	58.92
85	0.7369	14	1.0690	7.997	39	8.296	62.06
34	0.8263	14½	1.1470	8.578	40	8.727	65.28
31	0.9206	15	1.2270	9.180	41	9.168	68.58
61	1.0200	15½	1.3100	9.801	42	9.621	71.97
03	1.1250	16	1.3960	10.440	43	10.085	75.44
50	1.2340	16½	1.4850	11.110	44	10.559	78.99
03	1.3490	17	1.5760	11.790	45	11.045	82.62
63	1.4690	17½	1.6700	12.490	46	11.541	86.33
31	1.5940	18	1.7680	13.220	47	12.048	90.13
04	1.7240	18½	1.8670	13.960	48	12.566	94.00

* Actual.

apacity of pipes greater than those given, look in the table for a
f the given size and multiply its capacity by 4, or one of one-
d multiply its capacity by 9, etc. To find the WEIGHT of water
iven sizes, multiply the capacity *in cubic feet by the weight of a*
ter at the temperature of the water in the pipe (see page 1205).
city of a cylinder in U. S. gallons, multiply **the length by the**
eter and by 0.0034.

Cylindric

Diameter in feet and inche

1 g

Diam, ft	in	Area, sq ft*	Gal, 1-ft depth
1		0.785	5.87
1	1	0.922	6.89
1	2	1.069	8.00
1	3	1.227	9.18
1	4	1.396	10.44
1	5	1.576	11.79
1	6	1.767	13.22
1	7	1.969	14.73
1	8	2.182	16.32
1	9	2.405	17.99
1	10	2.640	19.75
1	11	2.885	21.58
2		3.142	23.50
2	1	3.409	25.50
2	2	3.687	27.58
2	3	3.976	29.74
2	4	4.276	31.99
2	5	4.587	34.31
2	6	4.909	36.72
2	7	5.241	39.21
2	8	5.585	41.78
2	9	5.940	44.43
2	10	6.305	47.16
2	11	6.681	49.98
3		7.069	52.88
3	1	7.467	55.86
3	2	7.876	58.92
3	3	8.296	62.06
3	4	8.727	65.28
3	5	9.168	68.58
3	6	9.621	71.97
3	7	10.085	75.44
3	8	10.559	78.99
3	9	11.045	82.62
3	10	11.541	86.33
3	11	12.048	90.13
4		12.566	94.00
4	1	13.095	97.96
4	2	13.635	102.00
4	3	14.186	106.12
4	4	14.748	110.32
4	5	15.321	114.61
4	6	15.90	118.97
4	7	16.50	123.42
4	8	17.10	127.95
4	9	17.72	132.56
4	10	18.35	137.25
4	11	18.99	142.02
5		19.63	146.88
5	1	20.29	151.82
5	2	20.97	156.83
5	3	21.65	161.93
5	4	22.34	167.12
5	5	23.04	172.38
5	6	23.76	177.72
5	7	24.48	183.15

Number of U. S. G
For On
1 cu f

Width, ft				
	2	2.5	3	3.5
2	29.92	37.40	44.88	52.36
2.5	....	46.75	56.10	65.45
3	....	....	67.32	78.54
3.5	....	....	....	91.64
4	....	....	....	....
4.5	....	....	....	....
5	....	....	....	....
5.5	....	....	....	....
6	....	....	....	....
6.5	....	....	....	....
7	....	....	....	....

Width, ft				
	7.5	8	8.5	9
2	112.21	119.69	127.17	134.65
2.5	140.26	149.61	158.96	168.31
3	168.31	179.53	190.75	202.97
3.5	196.36	209.45	222.54	235.63
4	224.41	239.37	254.34	269.30
4.5	252.47	269.30	286.13	302.96
5	280.52	299.22	317.92	336.62
5.5	308.57	329.14	349.71	370.28
6	336.62	359.06	381.50	403.94
6.5	364.67	388.98	413.30	437.60
7	392.72	418.91	445.09	471.27
7.5	420.78	448.83	476.88	504.93
8	....	478.75	508.67	538.59
8.5	....	....	540.46	572.25
9	....	....	....	605.92
9.5	....	....	....	.....
10	....	....	....	.....
10.5	....	....	....	.....
11	....	....	....	.....
11.5	....	....	....	.....
12	....	....	....	.....

To find weight of water in pound
by 8⅓.

Example. To find number of gal
10 ft., the water being 4 ft deep.
7.5 and opposite to this in the c
multiplied by 4, the depth of wa
gallons required.

(22) All joints must be made with picked oakum and molten lead and be made gas-tight. Twelve (12) oz of fine, soft pig lead must be used at each joint for each inch in the diameter of the pipe.

(24) Wrought-iron and steel water-pipes, vent-pipes, waste-pipes and soil-pipes must be galvanized.

(29) All brass pipe for soil-pipes, waste-pipes, and vent-pipes and solder-nipples must be thoroughly annealed, seamless-drawn, brass tubing of standard iron-pipe gauge.

Lead Waste-Pipes. (37) The use of lead pipes is restricted to the short branches of the soil-pipes and waste-pipes, bends, traps, and roof-connections of inside leaders. SHORT BRANCHES of lead pipe shall be construed to mean not more than

8 ft of 1½-in pipe
5 ft of 2-in pipe
2 ft of 3-in pipe
2 ft of 4-in pipe

(38) All connections between lead pipes and between lead and brass or copper pipes must be made by means of WIPED solder joints.

(39) All lead waste, soil, vent, and flush-pipes must be of the best quality known in commerce as *D*, and of not less than the following weights per linear foot:

Diameters	Weights per linear foot, lb
1¼ in (for flush-pipes only)	2½
1½ in	3
2 in	4
3 in	6
4 and 4½ in	8

(40) All lead traps and bends must be of the same weights and thickness as their corresponding pipe-branches. Sheet lead for roof-flashings must be 6-lb lead and must extend not less than 6 in from the pipe, and the joint made water-tight.

(41) Copper tubing when used for inside leader roof-connections must be seamless-drawn tubing not less than 22 gauge, and when used for roof-flashings must be not less than 18 gauge.

Yard, Area and Other Drains

(54) All yards, areas, and courts exceeding 15 sq ft in area must be drained into the sewer. A shaft open at the top and not exceeding 25 sq ft in area, and which cannot be connected in back of a leader, yard, court, or area drain-trap, may be drained into a publicly placed, water-supplied, properly tapped and vented slop-sink.

(59) These drains, when sewer-connected, must have connections not less than 3 in in diameter. They should be controlled by one trap, the leader-trap if possible.

Leaders

(60) Every building shall be kept provided with proper metallic gutters and rain-leaders for conducting water from all roofs in such manner as shall protect the walls and foundations of said buildings from injury. In no case shall the water from any rain-leader be allowed to flow upon the sidewalk or adjoining property, but the same shall be conducted by proper pipes to the sewer. If there be no sewer in the street upon which the buildings front, then the

l leaders shall be conducted by proper pipes below the surface of the
to the street-gutter, or may be conducted by extra-heavy cast-iron
leeching cesspool located at least 20 ft from any building. No plumb-
es shall discharge into a leeching cesspool.

side leaders must be made of cast iron, wrought iron, or steel, with
ections made gas-tight and water-tight by means of a heavy lead or
awn tubing wiped to a brass ferrule or nipple calked or screwed into

utside leaders may be of sheet metal, but they must connect with the
in by means of a cast-iron pipe extending vertically 5 ft above the
el.

eaders must be trapped with cast-iron running traps so placed as to
reezing.

ain-water leaders must not be used as soil-pipes, waste-pipes or vent-
r shall any such pipe be used as a leader.

e House-Sewer, House-Drain, House-Trap and Fresh-Air Inlet

he house-drain must properly connect with the house-sewer at a point
de of the outer front vault or area-wall of the building. An arched or
per opening in the wall must be provided for the drain to prevent
by settlement.

he house-drain if above the cellar-floor, must be supported at inter-
ft by 8-in brick piers or suspended from the floor-beams, or be other-
erly supported by heavy iron-pipe hangers at intervals of not more
t.

o steam-exhaust, boiler blow-off, or drip-pipe shall be connected with
-drain. Such pipes must first discharge into a proper condensing tank,
this a proper outlet to the house-sewer outside of the building must be
In low-pressure steam-systems the condensing tank may be omitted,
aste-connection must be otherwise as above required.

he house-drain and house-sewer must be run as direct as possible,
ll of at least ¼ in per ft, all changes in direction made with proper
nd all connections made with Y branches and one-eighth and one-six-
nds.

f House-Sewer. (74) The house-sewer and house-drain must be at
in diameter where water-closets discharge into them. Where rain-
charges into them, the house-sewer and house-drain up to the leader-
ns must be in accordance with the following table:

Diameter of pipe, in	For a fall of ¼ in per foot, sq ft of drainage-area	For a fall of ½ in per foot, sq ft of drainage-area
3	1 200	1 500
4	2 500	3 200
5	4 500	6 000
6	8 000	10 000
7	12 400	15 600
8	18 000	22 500
9	25 000	31 500
10	41 000	59 000
12	69 000	98 000

must be of the same diameter as the house-drain. An automat
by the superintendent of buildings may be used when set i
factory to him.

Note. The fresh-air inlet and running trap prescribed by S
are not required in many cities, and it is better to omit th
quired.

Soil-Pipes, Waste-Pipes and Vent-Pipe

(81) All main, soil, waste or vent-pipes must be of iron, st

(90) The diameters of soil-pipes and waste-pipes must not
given in the following table:

Main soil-pipes....................................
Main soil-pipes for water-closets on five or more floors..........
Branch soil-pipes..................................
Main waste-pipes..................................
Main waste-pipes for kitchen-sinks on five or more floors........
Branch waste-pipes for laundry-tubs..........................
When set in ranges of three or more..........................
Branch waste for kitchen-sinks..............................
Branch waste for urinals....................................
Branch waste for other fixtures..............................

(97) The SIZES OF VENT-PIPES throughout must not be les
ing:

For main vents, 2 in in diameter; for water-closets on thr
3 in in diameter; for other fixtures on less than seven floors,
3-in vent-pipe will be permitted for less than nine stories; fo
and less than sixteen stories, 4 in in diameter; for more tha
than twenty-two stories, 5 in in diameter; for more than twer

ot more than three wash-trays may connect with a single trap, an adjoining sink, provided both sink and tub waste-outlets de of the waste-line and the sink is nearest the line. When so ste-pipe from the wash-trays must be branched in below the

arge from any fixture must not pass through more than one ng the house-drain.

enware traps must have approved heavy brass floor-plates o the branch soil-pipe and bolted to the trap-flange and the ht. The use of rubber washers for floor-connections is pro- -flanges must be set in place and inspected before any water- n.

hall be placed at the foot of main soil- and waste-pipe lines. for traps must not be less than those given in the following

osets	4 in in diam.
ks	2 in in diam.
-sinks	2 in in diam.
ys	2 in in diam.
	2 in in diam.
baths	2 in in diam.
tures	1½ in in diam.

s, areas, floor and other drains must be at least 3 in in diam-

Water-Closets

ent-houses, lodging-houses, factories, workshops, and all he entire water-closet apartment and side walls to a height or, except at the door, must be made water-proof with asphalt, l, or other water-proof material as approved by the superin- s.

ral water-closet accommodation of any building cannot be r nor can any water-closet be placed outside of a building, n existing water-closet.

er-connected occupied buildings there must be at least one there must be additional closets so that there will never be persons per closet.

-houses there must be one water-closet on each floor, and re than fifteen persons on a floor, there must be one additional ery fifteen additional persons or fraction thereof.

sets and urinals must never be connected directly with or vater-supply pipes, except when flushometer-valves are used. ter-closet and urinal-cisterns and automatic water-closets s are prohibited unless approved by the superintendent of

r lining of water-closets and urinal-cisterns must not be lighter

set flush-*pipes must not be less than 1¼ in and urinal flush-* *ter, and if of lead must not weigh less than 2½* lb and 2 lb *uplings must be of full size of the pipe.*

test. All pipes must remain uncovered in every part
fully passed the test. The plumber must securely close
by the inspector of plumbing. The use of wooden pl
prohibited.

(172) The water-test will be applied by closing the
house-drain and filling the pipes to the highest openi
water. The water-test shall include at one time the ho
all vertical and horizontal soil, waste and vent and lead
therefrom to point above the surface of the finished flo
ished face of walls and partitions. If the drain or any
be tested separately, there must be a head of water at l
of the work so tested, and special provision must be
joints and connections in at least one test.

(173) After the completion of the plumbing-work,
building and before the building is occupied, a final smo
in the presence of the plumbing-inspector. Except that
six stories in height, a peppermint-test may be applied.

(174) The material and labor for the tests must be fur
Where the peppermint-test is used, 2 oz of oil of pepper
for each line up to five stories and cellar in height, and
oil of peppermint must be provided for each line when l
stories in height.

Traps

A trap is a device which permits the free passage of
also of any solid matters that may be carried by the li
time preventing the passage of air or gas in either dire
plumbing purposes are shaped so that an amount of wat
passage and prevent the passage of air will stand in th
principle of the common trap is shown in Fig. 7. The pi
from a sink or wash-basin, while the lower end *B* co

ion will be stopped, and the water in the pipe *T* will not fall below the the outlet at *b*. Several non-siphoning traps have been patented for ose of obviating the necessity of back-venting, but they are used to a tively limited extent. There are also several varieties of back-pressure

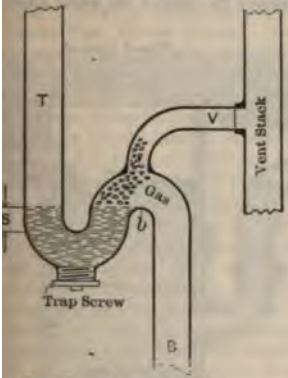

Fig. 7. Water-seal of Trap

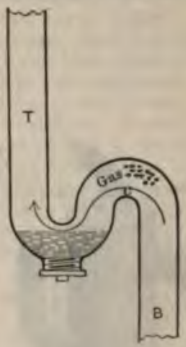

Fig. 8. Water-trap Unsealed

signed to prevent the sewage from flowing back into the house-drain. e in the nature of check-valves, and are used principally in seaport- here tide-water might possibly force the sewage back. The more com- pes of lead traps used in plumbing, with their trade names, are shown

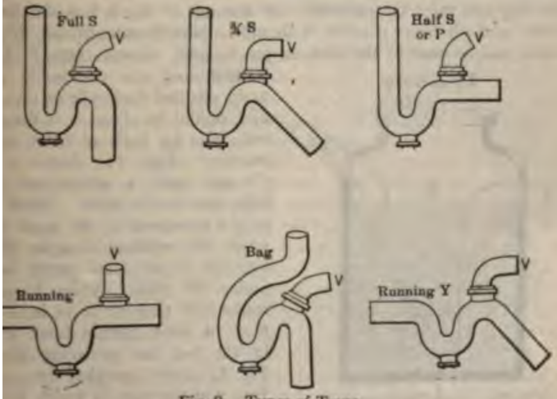

Fig. 9. Types of Traps

. The same shapes are also made of cast iron. The pipes marked *V* vent-connections. The drum-trap shown in *Fig. 10 has a deeper seal* e shown in *Fig. 9, and is commonly used under kitchen-sinks,* bath-tubs *-trays. Drum-traps are not easily siphoned, even when not vented, for water-closets are commonly formed in the fixture.*

Grease-Traps. The waste-water from kitchen-sinks always
siderable grease, which if permitted to enter the soil-pipe system is
the pipes by adhering to the walls. In certain localities grease giv
trouble than in others, due to the chemical composition of the wa
orado and many other places it is necessary to connect the waste f

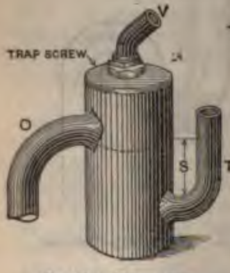

Fig. 10. Drum-trap

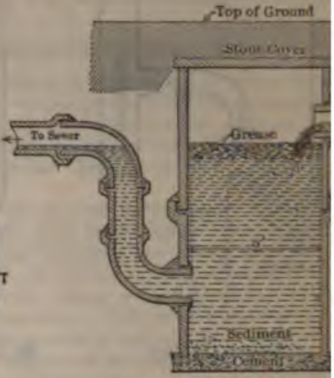

Fig. 11. Outdoor Grease-trap

sinks with a large grease-trap, which collects and holds the grease,
the water to pass into the sewer system. After a time the accum
fills the trap and must be removed. On account of this it is desir
large trap, and whenever possible it should be placed underground,
the house, and as near to the sink as practicable. Grease-traps

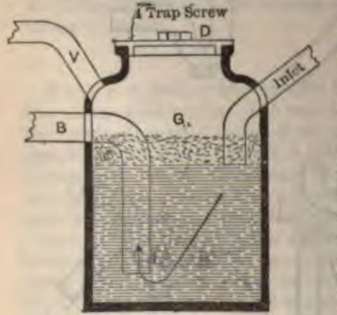

Fig. 12. Lead Grease-trap

underground are commo
24-in vitrified drain-tile or
and should be about 4 ft
may also be built of bric
mortar. Fig. 11 shows
through such a grease-tr
inlet and outlet-pipes. W
is in a basement or an up
when the building occupi
lot, the grease-trap mus
under the sink. When
round lead trap 12 or 14 i
may be used, with a larg
in the top for removing
Fig. 12 shows a section thr
trap and the way in which
tions should be made. A
of grease-trap is made of cast iron. Some city ordinances requir
grease-traps shall have a chilling-jacket for the purpose of m
separating the grease and thus preventing any of it from enterin
pipes. To be effective, a grease-trap must have a capacity of
the amount of greasy water that will be discharged into it at an

es. These may be of lead, brass, galvanized iron, tin-lined lead,
Lead pipe offers the least resistance to the flow of water, is easily
y situation, and easy curves are readily made. It is generally
re durable underground than galvanized-iron pipe. The grade
r STRONG, is the lightest that should ever be used, and when the
from city mains, in which there is a considerable pressure, *AA*,
pipe, should be used. Galvanized-iron pipe is probably more
d than any other material for water-supply pipes in buildings,
ickel-plated pipe is required, in which case brass piping is com-
rass pipe used for water-supply should be what is known as IRON-
ass piping is preferable to galvanized iron or lead for conveying
is largely used in the better class of buildings. Tin-lined iron
and pipes of block tin are usually considered as offering the
nce to corrosion or chemical action, and should always be used
le, beer and other liquors. Tin-lined iron pipe is made by pouring
a wrought-iron pipe. While in a fluid state the tin is inseparably
on, and the result is one solid pipe composed of two metals which
RN APART. It is essentially different from iron pipe merely
and immeasurably superior to iron pipe lined with a separate
will become detached. Its fittings are lined with tin to match.
not injure it, rats will not gnaw it, and thieves will not cut it
ot or cold water may stand in block-tin pipes and yet be drawn
e and free from poison or rust. Lead-lined pipe is made in the
insures delivering the water to the house just as it comes from the
ed by the chemical action which often results from contact with
ipe.

rawn Benedict Nickel Tubing is used to some extent for the
ing-pipes in high-class residences, office and public buildings.
ite metal throughout it cannot rub or wear BRASSY or become
is made in all the regular iron-pipe sizes, and necessary fittings
the same metal.*

ks. Where the pressure in the street-mains is not great enough
fficient volume of water for supplying the fixtures at all times, or
rivate water-supply, a tank should be placed in the attic, or ele-
6 ft above the highest fixture to be supplied. In some cases the
lower story are supplied direct from the street mains, while those
ory are supplied from a tank. The advantage of a tank is that it
lly from a very small stream, and thus form a reservoir from
volume can be drawn in a shorter space of time than could be
t from the service-pipes. Storage-tanks should always be pro-
overflow-pipe of ample size and when supplied from the street-
ly should be controlled by a ball-cock and float. Storage-tanks
ze are preferably made of wood lined with planished or tinned
lead, zinc or galvanized iron should not be used for lining tanks
ter for drinking or cooking purposes, and are not as durable as
hen the effect on the water need not be considered.

Tank Required will depend largely upon the character of the
s supplied from the street-main in which the pressure is fairly
not have a capacity exceeding 160 gal. Where the water is
he tank by a windmill or hot-air engine, the tank should have a
ent for a three or four days' supply at least.

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information consult the Benedict & Burnham Manufacturing Company,

Amount of Water Required for Various Purposes. Th
water required for household purposes has been found to be about
person, large or small, but waste will triple that amount sometim
will drink about 7 gal per day and a cow from 5 to 6 gal per day.
quires from 9 to 16 gal for washing.

Size of Supply-Pipes. The proper diameter of supply-pipes
several considerations, such as the number and size of faucets tha
be discharging water at the same time, the urgency of the demand,
the pipes and number of angles, and upon the pressure. There is
to having a pipe larger than is really necessary, except from the
cost. Service-pipes should always be one size larger than the tap
main. The following table affords a fair guide for proportionin
branches to plumbing-fixtures. If the pressure is less than 20 lb
system may be rated as LOW PRESSURE, and if above 20 lb as HIGH

Supply-branches	Low pressure, in
To Bath-cocks	¾ to 1
Basin-cocks	½
Water-closet flush-tank	½
Water-closet flush-valve	1¼ to 1½
Sitz or foot-bath	½ to ¾
Kitchen sinks	⅝ to ¾
Pantry sinks	½
Slop-sinks	⅝ to ¾
Urinals	⅜ to ¾

With high-pressure systems, dwellings of five or six rooms ar
for economy, supplied entirely through ¾-in pipe.

Minimum Diameter of Waste-Pipes. The following are
the smallest diameters allowable for waste-pipes. The diameter
New York City are given on page 1324.

Bath and sink-wastes, 1½ in.

Basin and urinal-wastes, 1¼ in.

Wash-trays, 1½ in from each compartment, entered into 4-in dr
2-in outlet from trap.

Water-closet trap, 2½ in.

Approximate Spacing for Tacks on Lead Pipes

Size of pipe, in	Vertical pipe		Horizontal
	Distance apart		Distance ap
	Hot, in	Cold, in	Hot, in
½	19	25	14
⅝	20	26	15
¾	21	27	16
1	22	28	17
1¼	23	29	18
1½	24	30	19

on of Lead Pipe. The different thicknesses of lead pipe were gnated by letters as in Table H, page 1332, but are now more signated as in Table G, following, which may be considered as pted by dealers.

Table G. Weights and Sizes of Lead Pipe

liber	Weight per foot		Caliber	Weight per foot	
	lb	oz		lb	oz
t	....	6	1½-in Aqueduct	3	....
ine	1	4	Extra light	3	8
uct	....	8	Light	4	....
ight	....	9	Medium	5	....
	....	12	Strong	6	....
n	1	....	Extra strong	7	8
	1	8	Extra extra strong	9	....
trong	2	....	1¾-in Extra light	3	12
uct	....	10	Light	4	8
ight	....	12	Medium	5	8
	1	....	Strong	6	8
n	1	4	Extra strong	8	....
	1	12	2-in Waste	3	....
	2	....	Extra light	4	....
trong	2	8	Light	5	....
xtra strong	3	....	Medium	7	....
uct	....	12	Strong	8	....
ight	1	4	Extra strong	9	....
	1	12	Extra extra strong	10	8
n	2	....	2½-in Waste	4	....
	2	8	Light	6	....
trong	3	....	Medium, 3/16 thick	8	....
xtra strong	3	8	Strong, ¼ thick	11	....
luct	1	....	Extra strong, 5/16		
light	1	8	thick	14	....
	2	....	Extra extra strong,		
m	2	4	⅜ thick	17	....
t	3	....	3-in Waste	4	....
strong	3	8	Light	6	3
extra strong	4	....	Medium, 3/16 thick	9	....
luct	1	8	Strong, ¼ thick	12	....
light	2	....	Extra strong, 5/16		
	2	8	thick	16	....
act	1	8	Extra extra strong,		
ight	2	....	⅜ thick	20	....
	2	8	3½-in Waste	5	....
m	3	4	Strong, ¼ thick	15	....
t	4	....	Extra strong, 5/16		
strong	4	12	thick	18	....
extra strong	5	8	4-in Waste	5	....
luct	2	....	Medium	10	....
light	2	8	Strong, ¼ thick	16	....
	3	....	Extra strong, 5/16		
m	3	12	thick	22	....
t	4	12	Extra extra strong,		
strong	6	....	⅜ thick	25	....
xtra strong	6	12	5-in Waste	8	....

Coils of supply-pipe weigh about 200 lb; aqueduct about 90
pipe, 100 to 180 lb each.

Block-tin pipe is stronger for a given weight per foot than lead
lined lead pipe. As compared with lead pipe its strength is as 3½

Tin-lined and lead-lined iron pipe is made with inside diameters
1¼, 1½ and 2 in, and in 10-ft lengths, threaded without couplings
and lead-lined fittings are also made (see page 1329).

Weights and Sizes of Sheet Lead

Thickness, in...	1/24	1/20	1/16	1/16 full	5/64	3/32	⅛	1/6	1/5	¼ full
Lb per sq ft.....	2½	3	3½	4	5	6	8	10	12	14

Table H. Thickness and Strength of Lead Pipes

Caliber, in	Mark	Weight per foot, lb oz	Thickness, in	Mean bursting pressure, lb	Safe working pressure, lb	Caliber, in	Mark	Weight per foot, lb oz	Thickness, in
⅜	AAA	1 12	0.18	1 968	492	1	A	4 0	0.21
⅜	AA	1 5	0.15	1 627	406	1	B	3 4	0.17
⅜	A	1 2	0.13	1 381	347	1	C	2 8	0.14
⅜	B	1 0	0.125	1 342	335	1	D	2 4	0.125
⅜	C	0 14	0.11	1 187	296	1	E	2 0	0.10
⅜	.....	0 10	0.087	1 085	271	1	.....	1 8	0.09
7/16	.....	0 9½	0.08	775	193	1¼	AAA	6 12	0.275
½	AAA	3 0	0.25	1 787	446	1¼	AA	5 12	0.25
½	.....	2 8	0.225	1 655	413	1¼	A	4 11	0.21
½	AA	2 0	0.18	1 393	343	1¼	B	3 11	0.17
½	A	1 10	0.16	1 285	321	1¼	C	3 0	0.135
½	B	1 3	0.125	980	245	1¼	D	2 8	0.125
½	C	1 0	0.10	782	195	1¼	.....	2 0	0.095
½	D	0 9	0.065	468	117	1½	AAA	8 0	0.29
½	.....	0 10	0.07	556	139	1½	AA	7 0	0.25
½	.....	0 12	0.09	625	156	1½	A	6 4	0.22
⅝	AAA	3 8	0.23	1 548	387	1½	B	5 0	0.18
⅝	AA	2 12	0.21	1 380	345	½	C	4 4	0.15
⅝	A	2 8	0.18	1 152	288	½	D	3 8	0.14
⅝	B	2 0	0.16	987	246	1½	.....	3 0	0.12
⅝	C	1 7	0.117	795	198	1¾	B	5 0	.....
⅝	D	1 4	0.10	708	177	1¾	C	4 0	.....
¾	AAA	4 14	0.29	1 462	365	1¾	D	3 10	0.125
¾	AA	3 8	0.225	1 225	306	2	AAA	10 11	0.30
¾	A	3 0	0.19	1 072	268	2	AA	8 14	0.25
¾	B	2 3	0.15	865	216	2	A	7 0	0.21
¾	C	1 12	0.125	782	195	2	B	6 0	0.19
¾	D	1 3	0.09	505	126	2	C	5 0	0.16
1	AAA	6 0	0.30	1 230	307	2	D	4 0	0.09
1	AA	4 8	0.23	910	227	...	....	.....	.....

Weight and Sizes of Pure Block-Tin Pipe

e	Weight per foot, oz	Size inside diameter in	Weight per foot, lb
	4	¾	9, 12, 16
	4, 5, 6	1	12, 16
	4, 5, 6, 8	1¼	20, 28
	4, 5, 6, 8	1½	24 and upwards
	5, 6, 8, 10	2	32 and upwards
	9, 12, 16	....	....

Sewer-Pipe

three kinds of sewer-pipe or drain-pipe offered in the market, (1) VITRIFIED CLAY PIPE, (2) SLIP-GLAZED CLAY PIPE and (3) CEMENT name of the latter sufficiently indicates what it is without any de- The SLIP-GLAZED CLAY PIPE is made of what is known as FIRE-CLAY, -brick clay, which retains its porosity when subjected to the most . It is glazed with another kind of clay, known as SLIP, which, when heat, melts, creating a very thin glazing, and which, being A FOREIGN TO THE BODY OF THE PIPE, is liable to wear or scale off. SALT-GLAZED made of a clay, which, when subjected to an intense heat, becomes glass-like. It is glazed by the vapors of salt, the salt being thrown hereby creating a vapor which unites chemically with the clay, and ing, which will not scale or wear off, and is impervious to the action es, steam, or any other known substance. It unites with the clay anner as to form PART OF THE BODY OF THE PIPE, and is therefore le. Salt-glazed pipe can only be made from clay that will vitrify, n subjected to an intense heat will become a hard, compact, non- . It should be borne in mind that SLIP-GLAZING is only resorted to ys are of such a nature that they will not vitrify.

erial of Drain-Pipes should be a hard, vitreous substance; not this would lead to the absorption of the impure contents of the drain, less actual strength to resist pressure, would be more affected by by the formation of crystals in connection with certain chemical s, or would be more susceptible to the chemical action of the con- the sewerage.

pes Should be Salt-Glazed, as this requires them to be subjected ore intense heat than is needed for SLIP-GLAZING, and thus secures a rial. Cement pipes made without metal reinforcement have not ciently strong and durable to be used with confidence in any im- . When reinforced with metal, however, they have ample strength, ed-cement sewer-pipes of large diameter are used to a considerable rope.

mining the diameter of house-sewers, the table on page 1323 will od guide. Storm-sewers should be proportioned to the area drained. mum rainfall, as shown by statistics, is about 1 in per hour, except heavy storms, equal to 27 225 gal per hour for each acre, or 453 gal per acre. Owing to various obstructions, not more than from 50 to rainfall will *reach the drain within the same hour,* and allowance *ide for this fact in determining size of storm-sewer* required.

Carrying Capacity of Sewer-Pipe

Gallons per minute

Size of pipe, in	Fall per 100 ft							
	1 in	2 in	3 in	6 in	9 in	1 ft	2 ft	3 ft
3	13	19	23	32	40	46	64	79
4	27	38	47	66	81	93	131	[illegible]
6	75	105	129	183	224	258	364	[illegible]
8	153	216	265	375	460	527	750	[illegible]
9	205	290	355	503	617	712	1 006	[illegible]
10	267	378	463	755	803	926	1 310	[illegible]
12	422	596	730	1 033	1 273	1 468	2 076	[illegible]
15	740	1 021	1 282	1 818	2 224	2 464	3 617	[illegible]
18	1 168	1 651	2 022	2 860	3 508	4 045	5 704	[illegible]
24	2 396	3 387	4 155	5 874	7 202	8 303	11 744	[illegible]
27	4 407	6 211	7 674	10 883	13 257	15 344	21 771	[illegible]
30	5 906	8 352	10 223	14 298	17 714	20 204	28 129	[illegible]
36	9 707	13 769	16 816	23 763	29 284	33 722	47 523	[illegible]

Quantities of Cement, Sand and of Cement Mortar for Sewer-Pipe Joints

Prepared by J. N. Hazlehurst

For each 100 ft of sewer (with Portland cement, 375 lb net per bbl)

Size of pipe, in	Length, ft	Mortar, cu yd	Proportions: 1 Cement to					
			1 Sand			2 Sand		
			Cement, bbl	Sand, cu yd	Pipe per bbl cement, lin ft	Cement, bbl	Sand, cu yd	Pipe per bbl cement, lin ft
6	2½	0.003	0.01248	0.00201	803	0.00855	0.00252	[illegible]
8	2½	0.038	0.15808	0.02546	633	0.10830	0.03192	[illegible]
10	2½	0.058	0.24128	0.03886	410	0.16530	0.04872	[illegible]
12	2½	0.089	0.37024	0.05963	270	0.25365	0.07476	[illegible]
15	2½	0.123	0.51268	0.08241	195	0.35055	0.10332	[illegible]
18	2½	0.167	0.69472	0.11189	144	0.47595	0.14018	[illegible]
20	2½	0.237	0.98592	0.15879	101	0.67545	0.19908	[illegible]
24	2½	0.299	1.24384	0.20033	80	0.85215	0.25116	117
27	3	0.492	2.04672	0.32964	49	1.40220	0.41328	[illegible]
30	3	0.548	2.27968	0.36716	44	1.56180	0.46032	[illegible]
36	3	0.849	3.53184	0.56883	29	2.41965	0.71316	[illegible]

Plumbing Specialties

The Kenney Flushometer. This is a gravity valve designed for flushing all water-closets, urinals and slop-sinks in a building direct from one tank situated in the attic or where most desirable, thus dispensing with the individual overhead tank. The pipe from the main tank is run down to the different floors either exposed or concealed and branches taken off from there to the ere are few cities in which the public water-supply is not greatly holesomeness by being filtered, and in many places filtering is ssary. The filter should be large enough so that the velocity of ng through it will be low and it should be so arranged that the an be reversed and the accumulated impurities washed into a ı the country a filter suitable for rain-water may be built un- filtering process being accomplished by beds of sand and gravel. ngs, however, a portable filter located in the basement should be nary sand filter, either pressure or gravity, will clarify water of impurities, suitable for plunge-baths, and other general uses in › provide a perfectly sterile water, however, the filter must be agulating apparatus to automatically feed a proportionate dose the raw water. Those so-called filters which are made to screw of an ordinary faucet should be considered merely as strainers, at purpose they soon become foul.

us Water-Heaters are a great convenience for heating water ash-basins in buildings in which a constant supply of hot water , and especially in residences where the cooking is done by gas. drical in shape, made of nickel-plated copper, and are usually lated shelf attached to the wall close to the fixture to be supplied. n in diameter and 30 in high will heat 20 gal of water in eight st of 1½ to 2 cts with gas at $1 per 1 000 cu ft. A large line of made by the Humphry Manufacturing and Plating Company, ch., for both gas and gasolene, although gas is preferable when The cost of heaters varies from $15 to $45, according to size.

tic Water-Heater which maintains water at any desired tem- t attention, provided the building has a supply of live steam, is B. Clow & Sons, the supply of steam being automatically regu- nostat. It will be found especially desirable in hospitals, hotels,

together. As long as the city water or steam passes thr
this suction and discharge continues. The supply of wa
on or off automatically, so that there is no consumption
except when the drainer is removing water. This drai
pressure of 15 lb or more, the heavier the pressure the
dead water discharged. When the drainage-water does
more than 10 ft, this is the most economical apparatus th
amount of city water consumed is very small. The Clim
six sizes, costing from $25 to $160.

Sewage-Ejectment. Mechanical ejectment of sewag
where the street-sewer is above the level of the area to b
dition is found principally in the subbasement-floors of
ground public-comfort stations and underground passeng
of mechanical ejectment consists of a gravity drainage-
tank or sump located in a water-tight pit at the lowest
system, and a pump or conpressed-air ejector to raise the
it into the street-sewer. There are three types of appara
age to the street sewers, centrifugal pumps, piston-pum
ejectors. The compressed-air ejectors, however, are com
their numerous advantages. They are automatic and al
tion, are perfectly odorless, and have but few working
of order. Sewage-ejectment apparatus is generally ins
that one set may be cut out of service for cleaning or
rupting the drainage-service.

Plunge-Baths

AN EXAMPLE OF THE CONSTRUCTION AND DETAILS
BATH OR SWIMMING-BATH. The following is a descripti
of the bath in the house of the Racquet and Tennis Club
New York City.*

"The swimming-bath has inside dimensions of 15 by
in total depth. It was built in a pit about 19 by 26 ft and
the main excavation, which was blasted out of solid ro
1 ft or more in thickness was laid over the bottom, s
which the 12-in walls of common red brick were laid i
built close to the rough vertical faces of the excavation,
them were filled with concrete or cement mortar or we
Then on the inner surface of the walls and on top of the
a waterproofing of six layers of felt with lapped joints w
tar and flashed around the iron outlet-pipe, which also
flange extending between the layers of felt. On the bott

-bath occupies one corner of the room and its elevated marble plat- nds entirely across it, forming a diving-platform which is reached by e steps. All the water-supply is filtered and it can be warmed by in- am into the delivery-pipe at the filter. The water enters through the rned end of a 2-in brass pipe projecting a foot or more through the the top of the bath and delivering a solid jet unless it is reduced by ting valve or is formed into a fan-shaped cascade by means of a

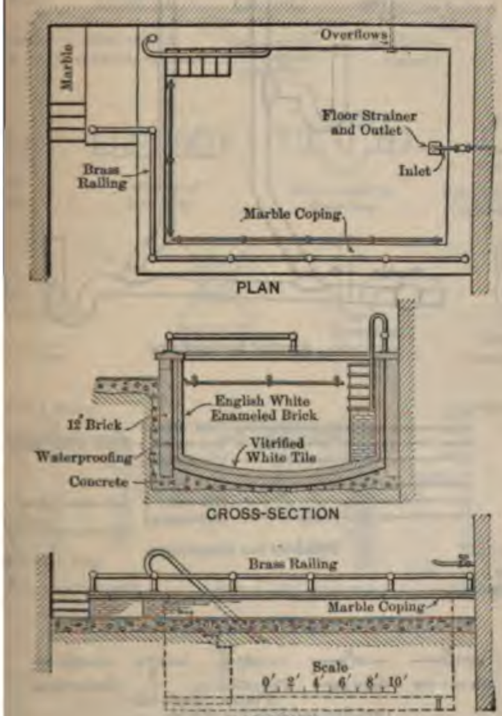
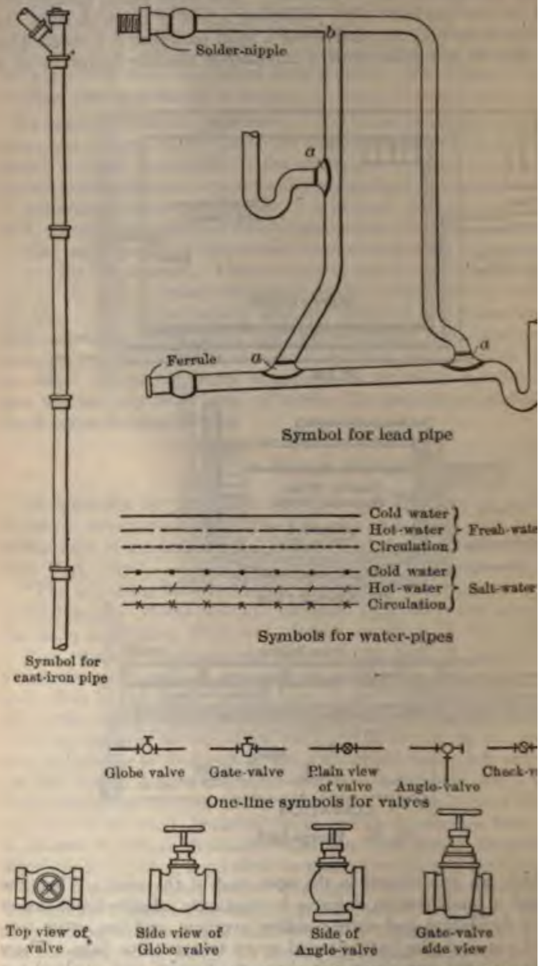


Fig. 13. Plunge-bath

zzle which can be screwed in the open end of the pipe. When the much used a small stream of water is constantly admitted and causes al gentle circulation and corresponding overflow, and the entire con- pumped out and the bath cleaned every two or three days. There overflows, an open one about 8 ft above the bottom and a valved one wer. C. L. W. Eidlitz was the architect of the house and *the water-* was done by the T. New *Construction Company."*

ls for Plumbing. Figs. 14, 15 and 16 show the symbols suggested *ing-Plans and Specifications "for designating* plumbing-work on plans and details, and generally accepted for the purpose. It is just to have conventional symbols to indicate plumbing-work and fi to have symbols to show-windows, doors, steps, partitions and



Symbol for lead pipe

Symbols for water-pipes

Symbol for cast-iron pipe

One-line symbols for valves

Top view of valve

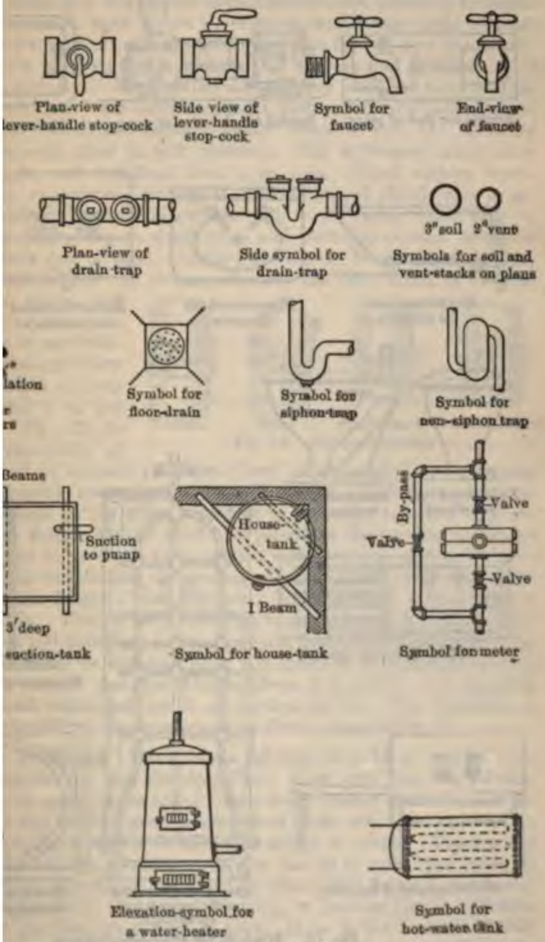
Side view of Globe valve

Side of Angle-valve

Gate-valve side view

Fig. 14. Symbols for Plumbing-pipes and Valves

tural details on architectural drawings. Before these symbols used there was no uniformity in the drawing of plumbing-p of standards often led to serious confusion. For instanc were examined, the chances were that on no two of them would ve been alike. Further, plans prepared in the same office at



Plan-view of ever-handle stop-cock

Side view of lever-handle stop-cock

Symbol for faucet

End-view of faucet

Plan-view of drain-trap

Side symbol for drain-trap

Symbols for soil and vent-stacks on plans

Symbol for floor-drain

Symbol for siphon-trap

Symbol for non-siphon trap

uction-tank

Symbol for house-tank

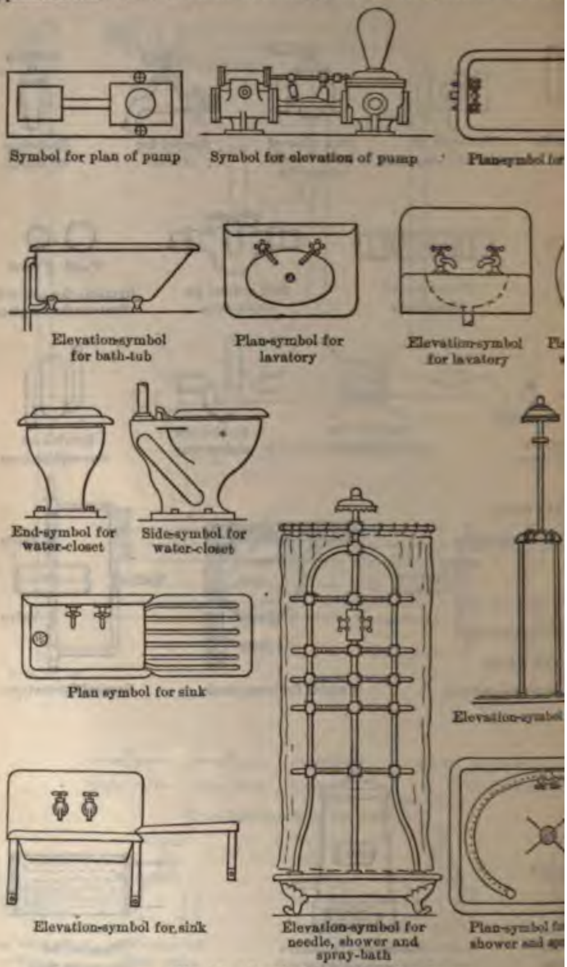
Symbol for meter

Elevation-symbol for a water-heater

Symbol for hot-water tank

Fig. 15. Miscellaneous Plumbing-symbols

or one set of *plans on which several different* draughtsmen had *ften show as many different symbols for a water-closet* or lava-*e workmen engaged on the drawings.* That was rather confusing to plumbers who had to take off quantities from the plan the symbols were so strange and bore so little resemblance to the apparatus that some of them were liable to be overlooked. It is ow



Symbol for plan of pump

Symbol for elevation of pump

Plan-symbol fo

Elevation-symbol for bath-tub

Plan-symbol for lavatory

Elevation-symbol for lavatory

End-symbol for water-closet

Side-symbol for water-closet

Plan symbol for sink

Elevation-symbol

Elevation-symbol for sink

Elevation-symbol for needle, shower and spray-bath

Plan-symbol fo shower and sp

Fig. 16. Symbols for Plumbing-fixtures

uncertainty wherever it exists from this cause, that there is a wid prices in the bids submitted, and all of them are unreasonably hi amount of work to be done. To avoid confusion and secure standard symbols should be used.

Soil and Waste-Stacks. In tall buildings, provision should
soil-stacks and connections to take care of the expansion, con-
ients, swaying and other movements of the building. This
inconsiderable amount, in some localities the settlement alone
much as 5 in when the foundations are not carried to bed-rock.
instance, most of the sky-scrapers which were built on com-
tion-beds are out of plumb and lean far out over the plumb-
ling in particular leaned so that the top was 30 in outside of
oundation. Most of the earlier heavy buildings there erected
ndations" are carried on jacks, and periodically jacked up as
rs. When the building finally comes to rest, the jacks are
e walls filled in with masonry. The settlement which takes
in such buildings from 3 to 5 in. These various move-
n, contraction, settlement, racking out of plumb, also sway-
ildings as they follow the sun in its course from East to
e destructive to steam-pipes and plumbing-pipes if provision
ake care of them. Steam-pipes always have expansion-loops,
ecently that the proper attention has been given to soil and
l pipes; and
as many as
s in one build-
through faulty
rigid connec-
edy is to put
(Fig. 17) in
ent-stacks of
nd to connect all water-closets to the soil-pipes by means
lapsible connections which will stretch, collapse, or stretch on
llapse on the other, according to the stress to which they are
se flexible fittings should be placed as close to the closets as
uld be used also in connection with slop-sinks and inside rain-
side rain-leaders the number of corrugations can be increased
the height of the building. Ordinary stock fittings have a
2 in. That is, they will stretch about 1 in and collapse 1 in.
in tall buildings, however, greater range than that is desirable.
s would be sufficient for a rain-leader in an ordinary building
n height; then, for taller buildings, it is well to allow an extra
each additional 100 ft or fraction thereof. The flexibility of
be seen in the accompanying illustrations of Fig. 17.

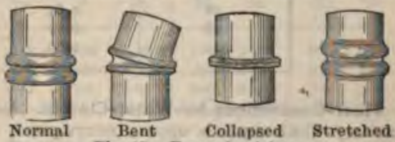


Fig. 17. Expansion-joints

Buildings. Ninety-seven per cent of buildings erected have
nstruction, and the floor-joists, when they dry out, shrink.
of many thousands of closets being broken annually, and the
e seal at the closet-connection of those which are not broken,
provided with a flexible floor-flange or fitting. The amount
floor-beams of different depths, can be found in the following
from information furnished by the United States Government,
Agriculture, Division of Forestry, in Bulletin No. 10. Besides
f the individual tiers of joists, there is the multiple shrink-
tiers when bearing-partitions, supporting the joists at the
ilding, rest on sills at each floor which are laid on top of
d of extending down through to the plate which supports the
hen the framing is properly done, there is only the shrinkage
beams to take into consideration. When improperly framed,

there might be three or four shrinkages affecting the top floor of the Even though the timbers are dry and seasoned when put in, by the plasterers are through the joists are wet and swollen from the moistu plaster and from the rain which saturates the timbers before the buildin closed. It is safe to assume, therefore, that a 12-in joist will shrink in, and an 18-in joist about ¾ in.

Table of Shrinkage of Timbers

Depth of green or wet timber, in	Amount lost by shrinkage, 4%, in	Depth of timber when dry, in
6	0.24	5.76
8	0.32	7.68
10	0.40	9.60
12	0.48	11.52
14	0.56	13.44
16	0.64	15.36
18	0.72	17.28
20	0.80	19.20

Floor-Connections for Water-Closets. No water-closet can be sanitary which depends upon a PUTTY-JOINT, slip-joint, rigid-gaske rigid connection of any kind for a seal. Improved metal-to-metal fl now cost no more than rigid-gasket joints formerly did, and they a water-tight, will remain permanently tight, and protect the closets f broken by shrinkage or other movement of the building or piping. way to get a perfectly sanitary water-closet is to specify a flexible, metal, closet floor-flange with it.

Expansion of Hot-Water Pipes. In all tall buildings expa ought to be placed in both the hot-water and the circulation-pipes, the expansion and contraction of the lines without injury to the syst loops are usually from 6 to 8 ft long, made up with elbows, and exte floor of the building. Generally the hot-water and circulation-pip ported midway between loops so that they can expand both up and d length that water-pipes will expand depends upon the degree to whi heated, and the materials of which the pipes are made. The first of ing three tables gives the expansion of cast-iron pipes, the second the of wrought-iron pipes, and the third the expansion of brass pipes.

Expansion of Cast-Iron Pipes

Temperature of air when pipe is fitted, degrees F.	Length of pipe when fitted, ft	Length of pipe when heated to					
		215° F.		265° F.		297° F.	
		ft	in	ft	in	ft	in
0	100	100	1.59	100	1.96	100	2.20
32	100	100	1.36	100	1.65	100	1.96
64	100	100	1.12	100	1.43	100	1.73

Expansion of Wrought-Iron Pipe

Length of pipe when fitted, ft	Length of pipe when heated to							
	215° F.		265° F.		297° F.		338° F.	
	ft	in	ft	in	ft	in	ft	in
100	100	1.72	100	2.21	100	2.31	100	2.70
100	100	1.47	100	1.78	100	2.12	100	2.45
100	100	1.21	100	1.61	100	1.87	100	2.19

Expansion of Brass Pipe

Length of pipe when fitted, ft	Length of pipe when heated to							
	215° F.		265° F.		297° F.		338° F.	
	ft	in	ft	in	ft	in	ft	in
100	100	2.58	100	3.18	100	3.56	100	4.05
100	100	2.19	100	2.79	100	3.18	100	3.67
100	100	1.81	100	2.41	100	2.79	100	3.28

Hard Water for Domestic Use. In many parts of the country TEMPORARILY HARD, PERMANENTLY HARD or both TEMPORARILY ENTLY HARD. This is due to the fact that in those regions the ock is limestone, and in percolating through the limestone the originally was soft, dissolves carbonates and sulphates of lime or m the rock. The solvent capacity of water for lime and magnesia ien the water is cold than when it is hot. Therefore, deep-well stone-regions is usually saturated with lime or magnesia, and when ter-tanks or boilers the point of saturation is lowered and lime is or liberated in the form of hard scale or incrustation. The effect istation is to shorten the life of the boiler and decrease the efficiency while in use. It is estimated that:

1/16-in lime-scale means a loss of 13% of fuel.
1/8-in lime-scale means a loss of 22% of fuel.
1/4-in lime-scale means a loss of 38% of fuel.
3/8-in lime-scale means a loss of 50% of fuel.
1/2-in lime-scale means a loss of 60% of fuel.
3/4-in lime-scale means a loss of 91% of fuel.

are probably a little high, but making due allowance, the table show the loss due to the use of hard water. In the laundry the isumption of soap to soften hard water is a further item of expense. ibout 1 lb of soap to soften 100 gal of moderately-hard water, soap required for washing after the water has been softened. expense, hard water forms an insoluble curd when washing which rticularly annoying to hotel-guests; therefore, it is advisable to d water *for large hotel-buildings, laundries and for many indus-* *Permanently hard waters contain sulphates of* lime or mag- *rarily hard waters contain carbonates of* lime or magnesia.

$W \div 10$ = sq ft of iron pipe-coil required for exhaus
$W \div 15$ = sq ft of copper pipe-coil required for exha
$W \times 0.07$ = sq ft of iron pipe-coil for 5 lb pressure-st
$W \times 0.045$ = sq ft of copper pipe-coil for 5 lb pressu
$W \times 0.05$ = sq ft of iron pipe-coil for 25 lb steam-pre
$W \times 0.035$ = sq ft of copper pipe-coil for 25 lb stea
$W \times 0.04$ = sq ft of iron pipe-coil for 50 lb steam-pr
$W \times 0.25$ = sq ft of copper pipe-coil required for 50
$W \times 0.03$ = sq ft of iron pipe-coil required for 75 lb
$W \times 0.02$ = sq ft of copper pipe-coil required for 75

Capacity of Water-Backs. The average size of wa
110 sq in, or about ¾ sq ft of exposed surface, will heat t
ature of domestic hot water, 180° F., about 21 gal of w
heat about 17 gal of water to the boiling-point with an
fire such as is used for roasting, washing, or baking, a wate
will heat about 23 gal of water to the boiling-point, or 2
of 180° F. Wrought-iron pipe heating-coils will heat fro
under the same conditions, and copper pipes will heat fro
for each square foot of surface exposed to the fire. In
capacity of water-backs or coils, the average temperatur
Thus, if water at 60° is heated to 200° F., the average te
would be $(60 + 200) \div 2 = 130°$ F., and the range of tem
it is heated would be $200 - 60 = 140°$ F.

Value of Pipe-Covering. Hot-water pipes and h
uncovered lose by radiation from their surface about 13
per square foot of surface. To prevent this loss of heat
consumption of coal, hot-water pipes, circulation-pipes a
large institutions are generally covered with some non-hea
The value of pipe-covering is not proportional to its thic

) ILLUMINATING-GAS AND GAS-PIPING*

of Gas. Five varieties of gas are now commonly used for light-
ng, namely:

s, which is made by heating bituminous coal in air-tight retorts.
ost common variety of gas furnished for the lighting of cities and

Gas, which is made usually from anthracite coal and steam, and
sively used in Eastern cities. Gas made by this process contains
an good coal-gas, and consequently does not give as bright a light,
rns perfectly in heating-burners. When used for lighting purposes
in carbon by vaporizing a quantity of petroleum by heat and in-
the hot gas before it leaves the generator. Pure water-gas is
s less odor than coal-gas.

Gas is obtained from holes or wells which are drilled in the ground.
here it can be obtained it furnishes cheap light and fuel. The
tained in the hard-coal regions develops more heat per cubic foot in
ny other kind of gas except acetylene. Natural gas is usually under
re in the street-mains and house-pipes than manufactured gas.

ne-Gas. Used almost exclusively for the lighting of isolated
or public buildings in towns or cities where there is no public gas-
ommonly generated on the premises. It is formed by bringing
lcium carbide in contact. Calcium carbide is produced by the
on of coke and lime. It is now a commercial article produced in
es and sold at a moderate price. It is a very hard substance like
has a very slight odor, will not burn or explode, and can be handled
y with perfect safety. The fact that carbide begins to disintegrate
cetylene at the slightest touch of moisture makes it practicable to
as in small quantities for single buildings.

Generating Acetylene-Gas. The satisfactory production of acety-
res a generator which shall feed carbide of sufficient size and weight
a sufficient depth under the water in the generator-chamber to
s and proper washing. The carbide-chamber must be so arranged
that no gas can return to it to be wasted when the chamber is
ermeate the house with its smell. It must feed carbide loosely
mall quantities, in order to provide for perfect coolness by free
er to all of the carbide. It must work automatically and with
inty. Acetylene-gas to be pure must be thoroughly washed.
lene, as with any other illuminating-gas, means a discoloration of
ninished illuminating power, clogging of pipes and burners with
ther foreign matter, and smoky burners, causing blackening of
arnished and soiled woodwork and upholstery. It is now gener-
hat the requirements above outlined can be attained only by a
he plunger-type. Portable generators which may be set in the
nent of any building are manufactured in great variety; it is esti-
0 000 acetylene-gas generators are now in use in the United States.
le in sizes of 5, 10, 15, 20 and up to 500-lights capacity. In all
pping carbide into water there should be a connection open from
olding receptacle to the safety-vent run out of doors from the
t is claimed that for a given degree of illumination, acetylene is
dollar gas. *A large residence may be lighted for about* $2.50 a

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e, also, Lighting and Illumination of Buildings, page 1351.

month. To develop the f
use a burner-tip having th
the gas being about fifteen
The light is a clear whit
fusiveness, with none of t
ordinary gas-flame, or the g
the quality, unique among
delicate shades of color as
burners, as it may be with
similarity from ordinary g
silver and copper, but does
ment specification for a co
Engineering News of Feb.

(5) **Gasolene-Gas** is a m
but is generated close to t
street stands, and the lik
changes from the liquid to t
at temperatures above 40
rapid at 70°, and furious
open to the air, the liquid
be lighted near an open ca
communicates the fire to t
Although generally consid
ignorantly handled. To p
about 4½ gal of the best
about 5 cu ft per hour.

Pi

General Principles and
as is used for steam or
Galvanized malleable-iron
superior. The coating of
covers all blow-holes, mak
of perishable cement. B
looked and blown through
caution will save much da
cement never should be us
and it can be dissolved by
metals is admissible for c
run across floor-timbers, t
supported on partitions, a
a 10-in timber notched 2
All branch outlet-pipes sho
Bracket-pipes should run u
center pipe from the bottom
side of the pipe. The who
traps, and decline toward th
as near the center of the bu
is distributed from the cent
needed than when the ma

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* Circular issued
† See, also, Ligh

p cutting, and the flow of gas will be more regular and even. For
on in large buildings, more than one riser may be advisable. When
s different heights of post, it is always better to have an in-
ing pipe for each height of post, than to drop a system of piping
er to a lower post, or to grade to a low point and establish
Drip-pipes in a building should always be avoided. The whole
ing should be so arranged that any condensed gas will flow back
system and into the service-pipe in the ground. All outlet-pipes
ecurely and rigidly fastened in position that there will be no possi-
moving when the gas-fixtures are attached. Center pipes should
d support fastened to the floor-timbers near their tops. The pipe
urely fastened to the support to prevent lateral movement. The
st be perfectly plumb, and pass through a guide fastened near the
e timbers, which will keep them in position despite the assault
asons and others. In the absence of express directions to the
lets for brackets should generally be 5 ft 6 in high from the floor,
is usual to put them 6 ft high in halls and bath-rooms. The upright
be plumb, so that the nipples that project through the walls will be
ipples should project not more than ¾ in from the face of the
Laths and plaster together are usually ¾ in thick; hence the
d project 1½ in from the face of the studding. Drop center pipes
t 1½ in below the furring, or timbers if there is no furring, where
hat there will be no stucco or centerpieces used. Where center-
be used, or where there is a doubt whether they will be or not,
p-pipes should be left about a foot below the furring. All pipes
ly fastened, the drop-pipe can be safely taken out and cut to the
when gas-fixtures are put on. Gas-pipes should never be placed on
of floor-timbers that are to be lathed and plastered, because they
ble in the contingency of leakage, or when alterations are desired,
res are insecure. The whole system of piping should be proved to
s-tight under a pressure of air that will raise a column of mercury
glass tube. The pipes are either tight or they leak. There is no
d. If they are tight the mercury will not fall a particle. A piece
ld be pasted on the glass tube, even with the mercury, to mark its
the pressure is on. The system of piping should remain under
ast a half-hour. It should be the duty of the person in charge of
tion of the building to thoroughly inspect the system of gas-fitting;
ch so as to inspect any other part of the building. He should know
al observation that the specifications are complied with. After
d that the mercury does not fall he should cause caps on the out-
osened in different parts of the building, first loosening one to let
ape, at the same time observing if the mercury falls, then tightening
ing the operation at other points. This plan will prove whether the
e from obstruction or not. When he is satisfied that the whole
erly and perfectly executed, he should give the gas-fitter a certificate
t.

ing requirements from specifications published by the Denver Gas
Company are worthy of attention. Always use fittings in making
t bend pipe. Do not use unions in concealed work; use long screws
-left couplings. Long runs of approximately horizontal pipe must
pported at short intervals to prevent sagging.

Rules and Table for Pro

Rules Governing Sizes
on the well-known formula f
and therefore the pressure n
the quantity of gas that goe
the loss in pressure not excee
increases in going from an ex
increasing number of outlets
be called on to pass throug
cubic feet, outlets being us
time of first inspection the n
In making the table, each ⅝-
cu ft per hour. In using the

(1) No house-riser shall be
extend from the cellar to th
pipe must be extended of the
taken off.

(2) No house-pipe shall b
may be made of ¼-in pipe t
pipe is not over 6 ft long.

(3) No gas-range shall be

(4) In figuring out the si
system, and work TOWARD th

(5) In using the table, th
lengths measured from one b
elbows or turns. Such leng
change in size of pipe may b
therefore being made of but

(6) If any outlet is larger
accordance with the schedule

Size of outlet, inches...
Value in table........

(7) If the exact number o
the next larger number.

(8) If, for the number of
feeds these outlets cannot b
sponding to the outlets give
quired. Thus, if there are ei
next larger than 55 in the ei
the 1¼-in column, that size

(9) For any given number
smallest size that contains
Thus, to feed 15 outlets, no
how short the section may be

(10) In any piping-plan,
meter, there may not be use
table for that size, as 50 ft fo
exceed the limit length, it m

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* The Denver Gas and Elec
† See, also, Lighting and Il
‡ With the exception of t
base, these rules are quote

le Showing the Correct Sizes of House-Pipes for Different Lengths of Pipes and Number of Outlets

ber -in ets	Lengths of pipes in feet									
	⅜-in pipe	½-in pipe	¾-in pipe	1-in pipe	1¼-in pipe	1½-in pipe	2-in pipe	2½-in pipe	3-in pipe	4-in pipe
	20	30	50	70	100	150	200	300	400	600
	....	27	50	70	100	150	200	300	400	600
	....	12	50	70	100	150	200	300	400	600
	....	....	50	70	100	150	200	300	400	600
	....	....	33	70	100	150	200	300	400	600
	....	....	24	70	100	150	200	300	400	600
	....	....	18	70	100	150	200	300	400	600
	....	....	13	50	100	150	200	300	400	600
	....	....	....	44	100	150	200	300	400	600
	....	....	....	35	100	150	200	300	400	600
	....	....	....	30	90	150	200	300	400	600
	....	....	....	25	75	150	200	300	400	600
	....	....	....	21	60	150	200	300	400	600
	....	....	....	18	53	130	200	300	400	600
	....	....	....	16	45	115	200	300	400	600
	....	....	....	14	41	100	200	300	400	600
	....	....	....	12	36	90	200	300	400	600
	....	....	....	....	32	80	200	300	400	600
	....	....	....	....	29	73	200	300	400	600
	....	....	....	....	27	65	200	300	400	600
	....	....	....	....	24	58	200	300	400	600
	....	....	....	....	22	53	200	300	400	600
	....	....	....	....	20	49	200	300	400	600
	....	....	....	....	18	45	190	300	400	600
	....	....	....	....	17	42	175	300	400	600
	....	....	....	....	12	30	120	300	400	600
	....	....	....	....	.....	22	90	270	400	600
	....	....	....	....	.....	17	70	210	400	600
	....	....	....	....	.....	13	55	165	400	600
	....	....	....	....	.....	.....	45	135	330	600
	....	....	....	....	.....	.....	27	80	200	600
	....	....	....	....	.....	.....	20	60	150	600
	....	....	....	....	.....	.....	.....	33	80	360
	....	....	....	....	.....	.....	.....	22	50	230
	....	....	....	....	.....	.....	.....	15	35	160
	....	....	....	....	.....	.....	.....	.....	28	120
	....	....	....	....	.....	.....	.....	.....	21	90
	....	....	....	....	.....	.....	.....	.....	14	59

e fed through 75 ft of 1-in pipe, but 1¼ in would have to be used. When
r more successive sections work out to the same size of pipe and their total
h or sum exceeds the longest length in the table for that size pipe, make the
n nearest the meter of the next larger size. For example, if we have 5 out-
o be supplied through 45 ft of pipe and these 5 and 5 more, making 10
, through 30 ft of pipe, we should find by the table that 10 outlets through
would require 1-in pipe, and that 5 outlets through 45 ft would also require
pipe, but as the sum of the two sections, 30 plus 45 equals 75 ft, is longer
the amount *of 1 in that may be used in any continuous* run, the 30-ft sec-
ing the one nearer the meter, must be made of 1¼-in pipe. The applica-

tion of the limit in length of any one size in a continuous run may as follows: Eight outlets will allow of 13 ft of ¾-in pipe in the se the eighth and ninth outlet (counting from the extremity of the s the meter), provided that this 13 ft added to the total length of may have been used between the extremity of the run and the does not exceed 50 ft, which, according to the table, is the grea ¾ in allowable in any one branch of the system. Therefore, up outlet, 37 ft of ¾-in pipe could have been used, and yet allow 13 ft used in the section between the eighth and ninth outlet. If mo had been used, then the entire 13 ft between the eighth and ninth have to be of 1-in pipe.

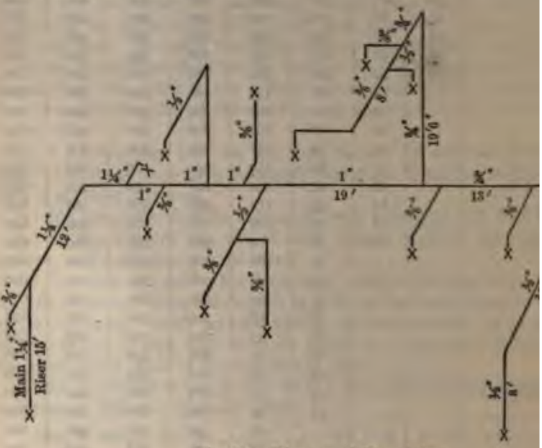

Fig. 18. Diagram of Gas-piping

(11) Never supply gas from a smaller size of pipe to a larger one. 25 outlets to be supplied through 200 ft of pipe, and these 25 and 5 30 in all, through 100 ft of pipe we should find by the table th through 200 ft would require 2½-in pipe, and 30 outlets through require 2-in piping, but as under this condition a 2-in pipe would a 2½-in pipe, the 100-ft section must be made 2½ in. The sizes of 18 are in accordance with the foregoing rules and the table.

ING AND ILLUMINATION OF BUILDINGS*

By

W. H. TIMBIE

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rinciples. Objects are illuminated for the sole purpose of making
o the eye. The eye, then, is the natural starting-point. When
the merits of any scheme of ordinary illumination, that which
it as a success or failure should be the general effect of the scheme
. Success should be measured largely by the degree of clearness
he objects are perceived by the eye, as to shape and color. If cer-
a room or street are too brilliantly lighted, objects in the dimmer
not perceived by the eye. If a certain side of one object is too
ated, the general shape of the object is lost, as the eye does not
ve its more dimly lighted parts. This is because the eye auto-
usts itself to the most brilliantly lighted area within its view, and,
s out of adjustment for perceiving the rest. We should get rid of
efore, that a light of intense brilliancy is the thing to be sought.
al, highly undesirable. A room may APPEAR brilliantly LIGHTED
ts looked at may not be sufficiently well ILLUMINATED for reading
g purposes. The lights APPEAR brilliant to the eye, but because
eir strongest rays in other directions than those in which they are
e, they do not give efficient illumination.

n between Light and Illumination. There is not only a great
ween LIGHT and ILLUMINATION, but there is a great difference be-
antly lighted room and a well-illuminated one. When anybody is
r a room is well illuminated or not, the chances are ten to one that
ks at the light itself. If the light appears to him to be brilliant and
vill invariably say, "Why, of course, the room is well lighted." He
ok away from the light at the objects around the room or under-
nt. If these can be seen clearly and easily, then the room is well
Afterwards he should look at the lights themselves, and if they ap-
pleasing to his eyesight the room is well LIGHTED. A room in which
ear soft to the eye and yet in which the eye can distinguish objects
n well lighted and well illuminated. A room in which the objects
o the eye while the lights remain dazzling is well illuminated but
A room in which the lights appear soft to the eye and the objects
uminated, is well lighted, but badly illuminated. A room in which
ear dazzling to the eye and the surrounding objects or those under-
not clear to the eyesight is both badly lighted and badly illuminated.
ood artificial illumination is to keep the illumination of objects as
ecessary, but the intensity or brilliancy of the lights as low as pos-
ng the first we enable the eye to see better; by doing the second we
e to feel better and suffer less from temporary discomfort or per-
. It is not generally understood that a light which is dazzling and
ne eyesight may not be giving as much illumination as another
t which appears soft, or even dim, by comparison. Thus an open
ore *dazzling than an enclosed light, but is less efficient in illumi-*

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also, Illuminating-Gas and Gas-Piping, pages 1345 to 1350.

and orange light in it. The sunlight contains waves of
thus is composed of all colors. The DIFFERENCE IN VIO
rise to a difference in intensity of the light. When the
object, they are partly reflected and partly absorbed.
as to the percentage of light they absorb and the per
two objects are illuminated by the same amount of ligh
the less light and reflects the more will appear the bright
light-waves of a certain length only, and absorb all th
erty that gives color to objects. Suppose, for instan
were receiving light from the sun, all of which it abso
proper length to cause a sensation of green to the eye.
would then come from the cloth to the eye, all the rest
cloth would appear green. If it absorbed waves of all
black, because no light would be reflected from it to th
of cloth, which absorbs all wave-lengths except that of
source of light which was emitting all colors EXCEPT GR
waves to be reflected from it, the cloth in this light wo
pose a piece of cloth absorbed all colors but two, say vio
having all wave-lengths fell upon it, it would absorb al
and red. These two, the cloth would reflect as a mi
purple. If, however, the source of light contained no v
reflect the red waves and appear red. This light, th
cloth to show its normal color. So in choosing an arti
necessary to select one which will send out all wave-le
the different objects appear in their normal colors.

Table I. Colors of Light-Sou

Sun (at zenith)	White (all colors)
Electric arc	Violet-white
Candle	Orange-yellow
Kerosene	Pale orange-yellow
Gas-flame	Pale orange-yellow

ıs shown that no artificial light except the CO_2 Moore tube is ›proximation to daylight. The Welsbach white mantle gives a ht than the tungsten-lamp, although neither can be said to light.

ty or Brilliancy. Candle-Power. The brilliancy of a source as its CANDLE-POWER; that is, the number of standard candles uivalent. Thus an ordinary open gas-flame, consuming 5 cu ft s equivalent in brilliancy to about 18 candles, and is said to have 8 candle-power. Welsbach lamps, consuming 3 cu ft per hour, 5 candle-power; that is, they are equivalent to 75 candles. ırces of light have the same amount of luminous surface, it is e a lamp by the number of candle-power per square inch of its jected) surface. Thus an ordinary candle-flame has about ⅓ l its intensity would be rated as 3 candle-power per square inch; lle-power it would have if its area consisted of exactly 1 sq in. led the INTRINSIC BRILLIANCY of a light-source.*

Accepted Values of Intrinsic Brilliancy for Various Light-Sources now in Use *

Light-Source	Candle-power per sq in
	0.3–1.75
incandescent-lamp	2–5
	3–4
	3–8
	3–8
amp	17
antle	20–50
er	75–100
ating-current arc-lamp	75–200
current arc-lamp	100–500
mps:	
atts per candle	375
atts per candle	480
s per candle	625
watts per candle	750
gsten 1.25 watts per candle	875
gsten, 1.0 watt per candle	1 000
tts per candle	2 200
	2 000
p	5 000
-filled	7 700
	10 000–50 000
	200 000
horizon	500 000
	600 000

* E. B. Rowe, Holophane Works.

llumination. Foot-Candle. The extent to which a surface is asured in FOOT-CANDLES. A surface has 1 foot-candle illumi-

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unt of light given out by a light-source is measured in LUMENS. For use of this term see any standard book on illumination.

nation when it is placed, at rig
1 candle-power intensity. T
power incandescent lamp wou

Law of Inverse Squares.
is held the less the illuminatio
far as stated above, it would
nation which an object recei
from the source. Thus, in t
much illumination, or 4 foot-
luminated by one-ninth of 16

Rule. To find the intensi
light-rays, divide the CANDLE
in FEET. The result will be
OF INVERSE SQUARES. Acc
illuminate a surface facing it
32 foot-candles, but a surface

Candle-Power and Foo
between CANDLE-POWER and
the intensity of a source o
intensity of illumination of s

Example 1. What is the
power lamp?

Solution. $\frac{32}{5 \times 5} = 1.28$ ft

Example 2. The illumina
about 2 foot-candles. (1) H
power lamp be hung? (2) A

Solution. $\frac{16}{x^2} = 2 \qquad x^2 =$

$\frac{32}{x^2} = 2 \qquad x^2 =$

The Primary Function
illumination as required by t
is devoted. The following t
tric power or of gas necessar
cluded.

Since the lower efficiencies
compensated by the lower i
the same watts per square fo
ditions are fairly favorable t
namely, light-cream or yellow
of average proportions with
High, narrow rooms may req
10% less, energy. Similar
respectively.

Three Systems of Ge
tion, as indicated in Tabl

III. Amount of Gas or of Electric Power Required to Illuminate Rooms Used for Various Purposes

Class of service	* Cu ft of gas per sq ft per hour	* Watts per sq ft
drill-hall	0.02 -0.025	0.5-0.6
n.	0.04 -0.05	1.0-1.3
p.	0.06 -0.07	1.5-1 7
e Auditorium)	....	....
oom	0.10 -0.112	2.5-2.8
eneral illumination)	0.01 -0.02	2.5-0.5
corridor)	0.016-0.02	0.4-0.5
operating-room)	0.14 -0.15	3.5-3.9
by)	0.06 -0.065	1.5-1.6
l-room)	0.05 -0.052	1.2-1.3
ing-room)	0.04 -0.045	1.0-1.1
aurant)	0.06 -0.07	1.5-1.7
chen)	0.05 -0.052	1.2-1.3
ting-room, general illumination only)	0.052-0.06	1.3-1.5
iard-room, general illumination only)	0.06 -0.065	1.5-1.6
fet)	0.065-0.072	1.6-1.8
eading-room)	0.055-0.06	1.4-1.5
tacks)	0.012-0.024	0.3-0.6
aking and accounting)	0.06 -0.065	1.5-1.6
eral)	0.052-0.06	1.3-1.5
vate)	0.05 -0.52	1.2-1.3
nographic)	0.06 -0.07	1.5-1.7
(bedroom)	0.012	0.3
(dining-room)	0.036-0.04	0.9-1.0
(hall)	0.008	0.2
(living-room)	0.036-0.04	0.9-1.0
(music-room)	0.02 -0.025	0.5-0.6
(kitchen)	0.05 -0.052	1.2-1.3
sembly or class-room)	0.04 -0.045	1.0-1.1
ss-room, business colleges)	0.055-0.06	1.4-1.5
ano, furniture, haberdashery, dry-goods, bile, clothing, cigar)	0.06 -0.07	1.5-1.7
ok, shoe, hardware)	0.055-0.06	1.4-1.5
s.	0.012-0.036	0.3-0.9

gures are based upon the use of Welsbach reflex lamps and Mazda electric Welsbach KINETIC lamps and nitrogen-filled tungsten-lamps (type C Mazda) 6 the values in the first and second columns, respectively. Data on gas R. F. Pierce, Welsbach Company.

t Lighting. A system is designated as DIRECT when more than one-ht reaches the area to be illuminated by coming directly from the , without being reflected from the ceiling or walls. This includes all ing lamps with clear, frosted, translucent, or opalescent globes, or n which the light is reflected downward. It is the most efficient sys-he first to be used, and is still the most common. The color of the ling has less effect in this system than in the others.

ect Lighting. A system is designated INDIRECT when all the light is t on *the ceiling and walls, and reflected from these to the* surface to be *Any system which conceals the source of light by* opaque reflec-*DIRECT. Light finish must always be used on the* walls and ceiling

with this system. Even then, th
system, but the total absence of
light make this a popular schem
rative lighting is desired.

(3) **Semiindirect Lighting.** Th
and ceiling, but allows a small
straight to the area to be illun
favor because apparently we hav
of light and miss it when it i
totally indirect fixtures often s
against a light background.* T
slightly higher efficiency of this s
Any given room may usually be
though it is generally true that
more desirable than either of the
detail how each system may be

General Consid

Outlets and Lamps. Outlets
possible square and equal areas i
lation, may be subdivided. The
the illumination and the greater
great care is used in planning the
minated surfaces, a disagreeable
The greater the height of lamps
the illumination. Figures sugge
ing-heights are given in Table IV

General Considerations i

Outlets and Lamps. The lo
the requirements for direct light
square and equal areas. Since gl
of the light is received from ceili
sible than in the case of direct lig
placed, the less uniform the illumi
the illuminating efficiency of the i
not be placed less than 2 ft from
fixture should be determined by
which will direct most of the lig
enough to illuminate the ceiling

A. The Interior Colorings an
of nearly white, cream, or light-
ward. Dark greens, reds, or blue
nation caused by a green color, o
60%. On the other hand, this sy
ations on the ceiling, and no colo
show dirt, where there may not b

---

* This unpleasant effect can someti
fixture.

† By R. F. Pierce, Welsbach Comp

‡ By G. S. Fobes, Macbeth-Evan

preferably should be matt, or satin, rather than glazed or var-
the matt ceiling-surface the maximum light will always be down-
varnished ceiling will reflect specularly, directing light sidewise or
images and glare.

nd details of decoration should be considered together with the
n, so that daylight-colors and reliefs will not be reversed or dis-
red light from artificial illuminants and shadows.

sitions of Outlets and of Fixtures. (1) Semiindirect units
ssible, be placed above the places where maximum light is

should not be so close to side walls as to cause light-spots running
icture-moldings, etc.
should be placed logically with reference to the ceiling-panels, so
brightly illuminated ceiling-areas will be the ones that on account
hapes, or decorations, will naturally bear emphasis. If the panels
beams), and one outlet is in each panel, it will ordinarily be
center. If several panels intervene between units, the fixtures
he beams rather than in the panels, to prevent dark ceiling-areas
of the beams.
should be such as to have the illuminated ceiling-areas overlap if
face is uniform.

oper Lamp and Bowl-Sizes. (1) Ordinarily the symmetrical
fixtures with respect to the other interior furnishings will largely
r sizes, although the bowls should never be so small as not to com-
and nearly surround the lamp-bulb.
ller the bowl and the brighter the lamp, the less effective the semi-
n becomes, and the more the effect approaches direct lighting.

and Styles of Bowls. (1) Bowls used close together or hung far
g should be of the focusing (upward) distribution, while broadly
owls are better when used singly, or when fairly wide apart and
iling.
o flat in shape may waste considerable light sidewise to the upper
efore be inefficient.
en-top bowls should not be used in halls, etc., where the bare
le to the observer from above, nor on or below the level of a bal-
nine.

Fixtures. (1) The average saving in light (expressed in terms of
) that results from washing once and dusting once monthly, will
ten times the cost of such cleaning. Bowls often collect films
are not visible and which materially reduce the efficiency both of
transmission.
with a dust-cap, button-ornament, or small area of thick glass at
ill allow dead insects or dirt to collect at that point without
pearance of the unit.
mmonia is an excellent glass-cleanser.
should be arranged to be lowered, for cleaning, from above if on a
ng in a church or similar structure.
d be possible to easily raise the lamp or lamps from within the
of dusting or wiping out.
estive of good practice in the selection of mounting-heights and
distribution are given in Table VI.

Table IV. Direct System

LAMP-SIZE, MOUNTING-HEIGHT AND SPACING*

Mounting-height	Commercial size of lamps in watts = W and cubic feet per hour		Watts per square foot = w and cubic feet per square foot		Ideal spacing = distance $\sqrt{\frac{W}{w}}$		Minimum spacing-distance	
ft	Watts	cu ft	Watts	cu ft	ft	in	ft	in
7 to 10	40	1.6	0.5	0.02	9	0	8	0
			1.5	0.06	5	2	4	6
			2.5	0.10	4	0	3	9
8 to 13	60	3.0	0.5	0.02	11	0	9	6
			1.5	0.06	6	4	5	6
			2.5	0.10	4	11	4	6
12 to 16	100	4.0	0.5	0.02	14	5	12	6
			1.5	0.06	8	2	7	0
			2.5	0.10	6	4	5	8
14 to 20	150	6.0	0.5	0.02	17	4	15	0
			1.5	0.06	10	0	9	0
			2.5	0.10	7	9	7	0
17 to 27	250	10.0	0.5	0.02	22	5	20	0
			1.5	0.06	12	11	11	9
			2.5	0.10	10	0	9	0
25 to 35	400	16.0	0.5	0.02	28	2	25	0
			1.5	0.06	16	4	15	0
			2.5	0.10	12	7	11	6
30 to 40	500	20.0	0.5	0.02	31	7	28	0
			1.5	0.06	18	6	16	6
			2.5	0.10	14	2	12	6

* To determine the size of equivalent Welsbach lamps allow 1 cu ft per h
25 watts. Adapted from the Electric Journal, by A. J. Airston.

The Designing of General Illumination by Each System
Tungsten or Welsbach Lamps

(1) From Table III should be determined the watts per square fo
for the given class of work, and the total number of watts necess
then be computed.

(2) From Table IV should be obtained the size of unit desirable
height of room and the number and spacing of fixtures then comput

(3) The ceiling should be laid off in squares the sides of which are
possible equal to the value of the ideal spacing. One fixture should
at the center of each square.

(4) Each lamp should be checked up on the plan to see that it is
clear of obstacles, and the layout incorporated into the building plan
standard methods and symbols for electricity or gas as the case may

Table V. Standard Symbols for Gas-Piping Plans*

Ceiling-outlet; gas only. Numeral indicates the number of single-mantle gas-lamps.

Single-lamp outlet (ceiling-units, pendants, etc.); gas only.

Ceiling-outlet; combination. $\frac{4}{2}$ indicates 4 electric lamps and 2 single-mantle gas-lamps.

Bracket-outlet; gas only. Numeral indicates the number of gas-lamps.

Bracket-outlet; combination. $\frac{4}{2}$ indicates 4 electric lamps and 2 gas-lamps.

Baseboard-outlet; gas only. Numeral indicates number of gas-lamps.

Floor-outlet; gas only.

Special outlet (for portable lamp, heater, etc.); gas only.

Outlet for outdoor-standard or pedestal; gas only. $\frac{2}{5}$ indicates 2 gas-lamps, with 5 mantles per lamp.

Outlet for outdoor standard or pedestal; combination. $\frac{6}{2/5}$ indicates 6 electric lamps, and 2 gas-lamps, with 5 mantles per lamp.

Arc-lamp outlet; gas only. Numeral indicates the number of mantles.

Pump or pneumatic lighting-system. Numeral indicates the number of lamps to be operated from one pump.

Push-button for magnet-valve. The numeral indicates the number of lamps to be operated from one push-button switch.

Meter-outlet.

Main or supply-pipe concealed under floor.

Main or supply-pipe concealed under floor above.

Main or supply-pipe exposed.

Branch pipe concealed under floor.

Branch pipe concealed under floor above.

Branch pipe exposed.

Street gas-main.

Battery-outlet.

Riser.

---

[*] Illuminating Engineering Laboratories, Welsbach Company.

Distance from Floor to Center of Wall-Outlets *

	ft	in
Living-room	5	6
Chambers	5	0
Offices	6	0
Corridors	6	3
Push-button switches or pneumatic pumps	4	0

* Illuminating Engineering Laboratories, Welsbach Company.

Examples of Design of Lighting-System for accounting-office, with 13-ft ceiling (Fig. 1). Walls and ceiling-light in color.

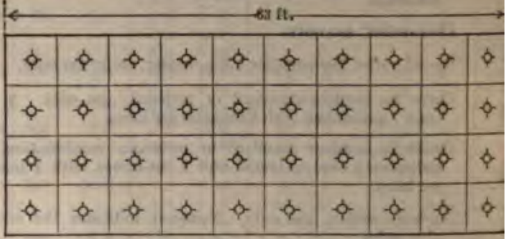


Fig. 1. Plan of Ceiling-lights

Direct System

Watts per sq ft = 1.5 (Table III)
Total watts = 1.5 × 63 × 25 = 2 400 (nearly)
Unit, size of = 60 watt electric (Table IV)
= 3 cu ft per hour, ordinary inclosed gas,
= 2 cu ft per hour, Welsbach KINETIC
Number of units = 2 400/60 = forty
Spacing (average) desired = 6 ft 4 in (Table IV)
Number of rows = 25/6⅓ = four
Number of outlets per row = 40/4 = ten
Spacing between rows = 25/4 = 6¼ ft
Spacing in rows = 63/10 = 6⅓ ft
Spacing-average = 6 ft 4 in.

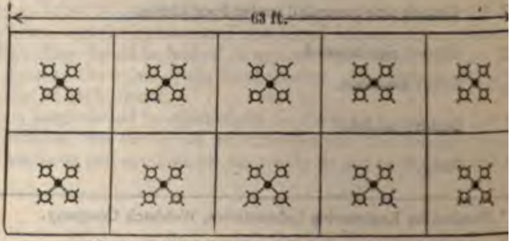


Fig. 1A. Modification of Plan Shown in Fig. 1

Fig. 1A is a modification of the plan shown in Fig. 1, and is a ment. It will not produce such even illumination but will result

, especially if fixtures are chosen which harmonize with the fur- he room. The lamps are placed in groups of four on ten fixtures equally spaced ne room. Here ays possible to higher wattage where the illu- not sufficient. ce of a proper lector is shown of Figs. 2 to be noted in Fig. tungsten-lamp greater part of he walls. The

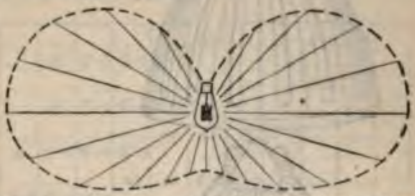
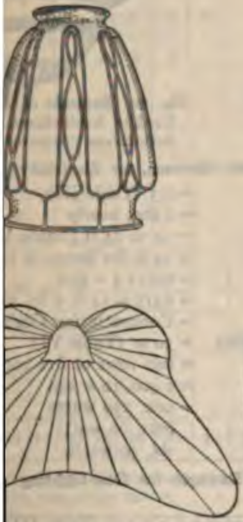


Fig. 2. Distribution of Candle-power about a Bare Tungsten Lamp

f any light can be controlled to a remarkable extent by the use


phane Reflector. Extensive tribution of Light

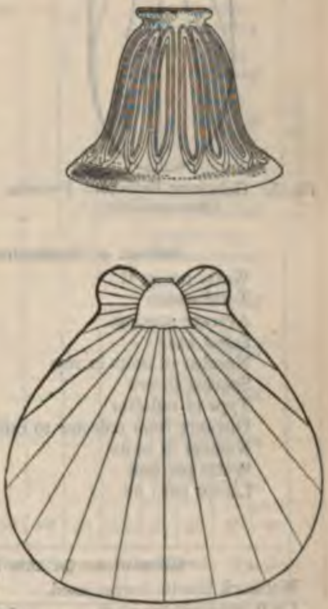


Fig. 4. Holophane Reflector. Intensive Distribution of Light

reflector. Figs. 3 to 5 show how the several types of Holophane ribute *the light.*

---

* *Furnished by E. B. Rowe, of the Holophane Works.*

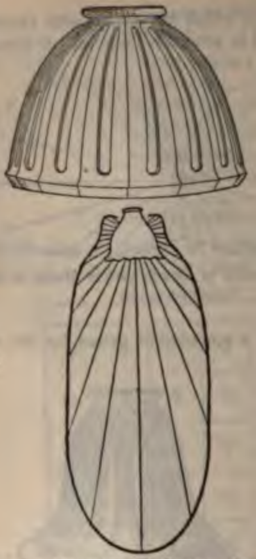


Fig. 5. Holophane Reflector. Focusing Effect on Light

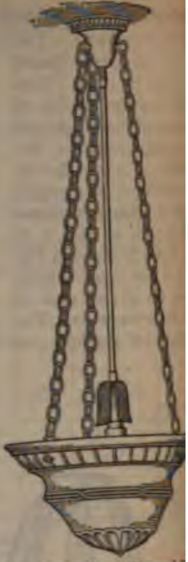


Fig. 6. Example of Type of
Used in Semiindirect System
beth-Evans Company

Indirect or Semiindirect Systems, for Electricity

Watts per sq ft = 1.5 (Table III)
Total watts = 2 400, nearly
Average spacing = 14 to 24 ft (Table VI)*
Select 25/2 = 14 ft for lamps in two row
Number of units in row = 63/12.5 = five
Spacing in row = 63/5 = 12 ft 7 in, about
Type of reflector = Concentrating (Table VI)
Distance from reflector to ceiling = 30 in (Table VI)
Number of units = two rows of five each = te
Watts per unit = 2 400/10 = 240
Lamps per unit = one, 250 watts
four, 60 watts
six, 40 watts

Calculations for this Example for Gas-Lighting

Welsbach kinetic burner used.
Cu ft per hour per sq ft = 0.06 × 0.6 = 0.036 (Table III)
Total hourly consumption = 63 × 25 × 0.036 = 57 cu ft per hour.
Average spacing (see above) = 12½ ft
Number of units = ten
Consumption per unit = 57/10 = 6
Reflector and mounting-height as in preceding problem
Lamps per unit = one, 6 cu ft
Lamps per unit = two, 3 cu ft, etc.

---

* See How to Use Table VI, immediately following the Table.

'or Determining Number of Ceiling-Outlets, Type of Reflector d the Distance from Top of Reflector to Ceiling for Indirect and Semiindirect Lighting *

																Type of reflector
ributing					....	....	....	....	....	....	48	48	48	5′-0	6′-6	
entrating					....	....	48	5′-0	6′-6	7′-0	....	....	....	....	....	
ributing					....	....	....	....	....	....	48	48	5′-0	6′-0	....	
entrating					....	....	48	6′-0	6′-6	6′-6	....	....	....	....	....	
ributing					....	....	....	....	....	42	48	48	6′-0	....	....	
entrating					....	42	5′-0	6′-0	6′-0	....	....	....	....	....	....	
ributing					....	....	....	....	42	42	4′-6	6′-0	....	....	....	
entrating					36	4′-6	6′-0	6′-0	....	....	....	....	....	....	....	
	....	....	....	....	....	....	....	42	42	4′-6	5′-6					Distributing
	....	....	....	36	42	5′-6	5′-6	....	....	....	....					Concentrating
	....	....	....	....	....	....	30	36	4′-6	5′-0	....					Distributing
	....	....	30	36	4′-6	5′-0	....	....	....	....	....					Concentrating
	....	....	....	....	....	30	30	48	5′-0	....	....					Distributing
	....	....	30	48	5′-0	....	....	....	....	....	....					Concentrating
	....	....	....	....	30	30	48	....	....	....	....					Distributing
	....	30	36	48	....	....	....	....	....	....	....					Concentrating
	....	....	....	24	30	42	48	....	....	....	....					Distributing
	....	30	48	....	....	....	....	....	....	....	....					Concentrating
	....	....	24	24	36	48	....	....	....	....	....					Distributing
	24	36	....	....	....	....	....	....	....	....	....					Concentrating
	....	....	24	30	42	48	....	....	....	....	....					Distributing
	24	42	....	....	....	....	....	....	....	....	....					Concentrating
	....	....	24	36	42	....	....	....	....	....	....					Distributing
	30	42	....	....	....	....	....	....	....	....	....					Concentrating
	....	18	24	36	....	....	....	....	....	....	....					Distributing
	30	....	....	....	....	....	....	....	....	....	....					Concentrating
	18	18	30	36	....	....	....	....	....	....	....					Distributing
	....	....	....	....	....	....	....	....	....	....	....					Concentrating
	18	24	30	....	....	....	....	....	....	....	....					Distributing
	....	....	....	....	....	....	....	....	....	....	....					Concentrating
	18	24	....	....	....	....	....	....	....	....	....					Distributing
	....	....	....	....	....	....	....	....	....	....	....					Concentrating
	18	....	....	....	....	....	....	....	....	....	....					Distributing
	....	....	....	....	....	....	....	....	....	....	....					Concentrating
	2	14	16	18	20	22	24	26	28	30	32	34	36	38	40	

de of the limiting square in feet that can be uniformly illumi-
m one center outlet

es the distance in inches (except as noted) from the top of the reflector to
btain the desired distribution of light from one ceiling-outlet.
s are not given to the left, it is advisable to submit data to illuminating
or greater ceiling-heights than 20 ft.

---

* H. B. Wheeler, X-Ray Eye-Comfort Company.

An idea of the appearance of some of the typical modern fixtures
or electricity in these systems of lighting may be obtained from
and 8.

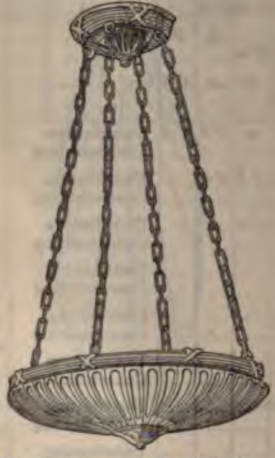


Fig. 7. Type of Fixture in Indirect Illumination. National X-Ray Reflector Company

Fig. 8. Fixture Used for G
Either Indirect or Semi
System

How to Use Table VI. In the first column of Table VI is located t
of ceiling, in this case, 13 ft. The last square to the right of this figu
has a number in it, is noted. In this case the last square to the right
which has a number in it, contains the number 48. By following thi
containing the number 48 down to the figures printed in heavy type at th
of the table, the heavy-faced number in this case is found to be 24.
the length of the side of the largest square which a single fixture can pro
minate when the ceiling is 13 ft high. The 48 which is opposite the 13
the number of inches the fixture must be hung from the ceiling.
largest squares into which we can possibly divide the ceiling have 24
But a room 25 ft wide cannot be divided into 24-ft squares. We are
therefore, to divide it into squares of a smaller size, since the fixtures
illuminate any larger square. The greatest length into which we c
25 ft is 12½ ft. We may, then, decide to use fixtures which will illumin
squares. Locate the number 14 in heavy type at bottom of table, and
the column in which it is found until the square is reached which is op
ceiling-height of 13 ft. Here the number 30 is found. This means
hang the fixture so that the top of it is 30 in from the ceiling in order
desired results. Looking along the squares to the right of the one in
find the 30, we find the word CONCENTRATING, which signifies the type of
advised for this installation.

are required.

ng Shops. The surfaces here are generally high and offer good rties, so that 1½ watts per sq ft, direct system, are sufficient.

ps. Because the belts, machines and dingy floors offer great ces at least 2 watts per sq ft, direct system, are necessary.

The dark molding-sand and the dust and smoke in the air make ft, direct system, necessary.

ms. The semiindirect system with 2¼ watts per sq ft (about of 1¾ watts per sq ft, direct system) has proved highly satis-

by Gas.† Recent progress in incandescent gas-lighting has development of appliances in which practically all of the short- vious types are overcome, and except for inaccessible locations, are very infrequently lighted, there is little to choose between the , gas and electricity, upon the score of convenience or of artistic ile the greater economy of gas-lighting (often in the ratio of about led with the freedom from interruption which characterizes gas- it desirable to pipe all buildings, particularly residences, for gas eferably installing combination-fixtures and providing wall-out- oard-outlets for the connection of the various gas-operated con- h are being developed in rapidly increasing numbers.

tic-Burner Lamps [With Nearest Equivalent Sizes in Electric Incandescent Lamps

Lamps	Mazda watts	Nitrogen-filled Mazda (Type C) watts

Lamps in inaccessible positions.
Lamps lighted at infrequent intervals.
Lamps placed very close to ceiling (12 in or less)
Poor gas service as regards:
  Pressure-regulation (more than 50% variation
  Non-uniformity of gas-quality.
  Imperfect purification.
Good electric service as regards:
  Voltage-regulation.
  Freedom from liability to derangement by acc
Non-rigid fixtures.

(2) Factors favorable to the use of gas:
Units of 60 candle-power or more.
Accessible locations.
Frequent use of lamps.
Lamps placed 15 in or more from ceiling.
Good gas service as regards:
  Pressure-regulation (not more than 50% varia
  Uniformity of gas-quality (chemical compositi
  Proper purification.
Poor electric service as regards:
  Voltage-regulation (more than 5% variation
    likely on alternating-current circuits.
  Liability to derangement by accident (overhe
  Rigid fixtures.

Hygiene.* From the hygienic point of view there i
the two illuminants. The investigations of Dr. Rideal h
light positively improves the air for breathing purposes
tions of use. The causes of this improvement are th
tion, the destruction of disease-germs and the additio
Gas-burners give rise to stronger air-currents and in
active ventilation and diffusion of air than electric li
the products of the gas-burner the exhalations of p

the same direction, namely, that gaslight, as used in these
less fatiguing to the eye than electric light." Under semi-
ect lighting, of course, no such disparity in effect is found.

ıg Rules Indicate the General Practice in planning the illu-
m. It must be said, however, that this set of rules must not be
ishly. In illumination no rules can take the place of judgment
Each project must be considered more or less as a problem by
previous experience and former installations should be made to
to suggest methods. It is well, therefore, when planning the
room, to visit as many similar rooms as possible, note the effect
use and obtain data as to their efficiency, cost, etc. The most
e may then be used as the basis for planning the desired instal-

he Diffusion of Light through Windows *

e Diffusion of Light by Glass. Abstracts from report of
n, on an elaborate series of tests made at the Massachusetts In-
ology:† The results of the tests on a score or more of different
tated briefly. We may increase the light in a room 30 ft or
om three to fifteen times its present effect by using FACTORY-
stead of PLANE GLASS in the upper sashes. By using prisms
certain conditions, increase the effective light to fifty times its
. The gain in effective light on substituting ribbed glass or
glass is much greater when the sky-angle is small, as in the case
ning upon light-shafts or narrow alleys. The increase in the
light directly opposite a window in which ribbed glass or
substituted for plane glass is at times such as to light a desk or
the window better than one 20 ft from the window had pre-
ted.

Glass Tested were as follows:
ass of different degrees of fineness.

(7) Ripple-glass, with rippled surfaces on both sides; of very beautifu
ance and a clear white color.

(8) Glass ribbed on one side and figured on the other.

(9) Ribbed glass with a wire net pressed into it, to increase its resistan

Of these several specimens, one or two may be dismissed with brie
Ground glass is of little value, except as a softening medium for bright

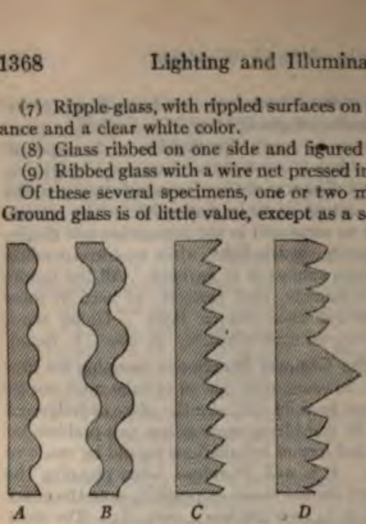


Fig. 9. Types of Ribbed or Prism-glass

Its rapidly increasing o
with moisture and dust
undesirable as a wind
The common rough plat
little action as a diffusing
giving no perceptible
the effective light. Ri
has great value as a
medium in small roo
nearly open horizon. Of
glasses, the fine Factor
with twenty-one ribs to
is distinctly the best,
probability because of
ness, but because of t
sharpness of the co
The Ribbed wire-glass is about 20% less effective than the ordinary
Ribbed glass. The addition of a second corrugation upon the back o
giving the Skylight and Wash-Board glass is of no apparent value.
pattern imprinted upon one surface of the glass, as in the case of
glass, gives the widest diffusion, especially in bright sunlight. A ra
when worked upon the back of the Ribbed glass, renders it less offen
eye in bright sunlight, but less effective in deep rooms. The only
this group which it is worth while, then, to discuss further are th
Ribbed and the Maze glass.

The second group comprises the following glasses:

(1) The Luxfer prisms.
(2) The Solar prisms.
(3) The Daylight-prisms.
(4) The glass of prismatic section made by the Mississippi Glass C
(5) Three-way prisms.
(6) Maltby prisms.

The Luxfer prism consists of a plate smooth on one side and dee
on the other as in *C*, Fig. 9, the teeth or prisms being of very fl
faces of brilliant appearance. The glass is clear white, and the pris
canopies and in the major part of the vertical glazing are made in til
about 4 in square. Tiles are built up in large sheets in frames of copp
so made as to give to the sheets of tiles a strength and durability f
of a single sheet of the same size. The Luxfer prisms are made f
use in large sheets, as well as in the small tiles. The Solar prisms a
small tiles, which are held together in a metal frame to make large sh
main difference between the Solar and Luxfer prisms is that the under
former prism is curved instead of plane, as in *D*, Fig. 9. The Dayli
tested were made in large sheets and of approximately the same cross-
general appearance as the Luxfer prisms for factory-use. No tiles of
prisms were tested, as none came to hand in time for the test. The
prism glass is much like the other prisms in cross-section, but the

across the plate in a straight line, but in a wavy or sinuous line. ising from this over the straight-edge prism was detected.

1) The conditions in a room less than 15 ft deep are such a skylight of less than 45°, it is not advisable to alter the general t by using a prismatic or ribbed glass. A nearly hemispherical is given by the -glass, is ordi- om is from 20 even more, and 60° or less, the tic glass results ain in effective in brilliancy is basement with s light as a plane glass. ndows opening s and narrow mited openings e the available , may have the light 20 ft back from the window increased ten by using prisms; and, by using canopies of prisms, it is some- strengthen the light fifty to one hundred times. With sky- less, and in deep rooms, the relative efficiency of the prism tly. The refraction of the incident-ray in a case of the ribbed glass and prism is shown in Fig. 10. Ribbed and maze, glass are of very great value in softening the light, especially in the case of such windows as are exposed to the direct sun, aside from their effectiveness in strengthening the light at distant points. With the Maze glass, the artist may have, in all weather and in all directions, what is in effect a much-desired NORTH LIGHT. The photographer may have in this way as well diffused a light as he now has with cloth screens or shades, and with a much greater intensity. To be efficient in rooms 20 ft deep or more, ribbed glass should be set with its ribs horizontal, and where the sunlight falls upon it, it should be provided with thin white shades. All inferences drawn from the test are made upon the assumption that the windows are to be glazed with diffusing glass only in the upper half, which is the common practice. If the lower sash is to be glazed lass as well, a further increase of about 25% may be

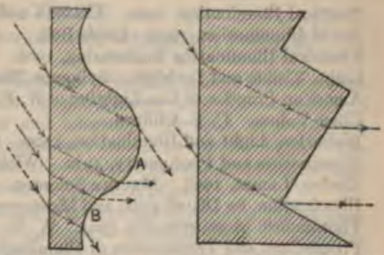


Fig. 10. Refraction of Light in Ribbed and Prism-glass

t and First Story
m Court

th expense and efficiency, the following general suggestions are or Ripple-glass in small rooms or offices not more than 15 or 20 ft -Ribbed glass in rooms from 30 to 50 ft deep, with sky-angles of prisms or Factory-Ribbed glass, in sheets, in all vertical win-

Radiation, Light and Illumination, 1909. C. P. Steinm
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ELECTRIC WORK FOR BUILDINGS

By
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APPLIED SCIENCE DEPARTMENT, WENTWORTH INSTITUTE

siderations and Definitions. Electrical energy is now in com- hing power, heat and light, operating bells and buzzers, and ssages by telephone and telegraph. In order to accomplish current of electricity must flow around an electric circuit. The city is not known, but the flow of it through an electric circuit he flow of water through a system of pipes.

nperes. The flow of water is measured in GALLONS PER SECOND. tricity is measured in AMPERES. An ampere-flow of electricity a gallon-per-second flow of water. The amperes thus indicate electricity flowing through an electrical appliance in one second. e is flowing through an ordinary carbon-filament incandescent is glowing at r. The same npere causes a n lamp to pro- candle-power. ually requires to 10 amperes.

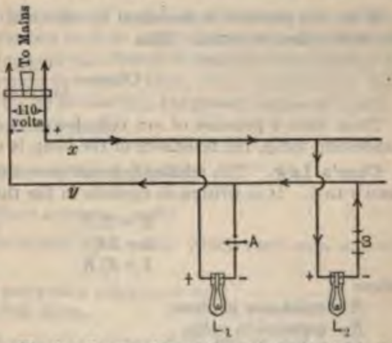


Fig. 1. Current Always Flows from (+) to (−)

olts. When a er flows from ther in a pipe- lways because aulic pressure it to flow. usually meas- ls per square , when a cur- ity flows from nother in an it is because there is an electric pressure present which w. This electric pressure is measured in VOLTS. An electric LT is analogous to a hydraulic pressure of 1 lb per sq in. The causes the ½-ampere current to flow through an incandescent 110 volts. The electric company instals at least two wires in a hen maintains an electric pressure of 110 volts between them ter company maintains a pressure in the water-pipes. This is at all times tending to force electricity from one wire to the ss the space between the two wires, just as the water-pressure he water out from the pipe. The rubber insulation is put on to w, very much as the strength and compactness of the iron pre- f water through the walls of the pipe. But when one terminal nected *to one wire and the other terminal to the other* wire, the *tending to send a current from one wire to the other,* sends a *he lamp and causes it to glow. We mark the wire bringing the*

Resistance. Ohms. The fact that a current of only an incandescent lamp when a pressure of 110 volts is a RESISTANCE of the fine filament. This resistance of t to the resistance which a pipe of small bore offers to resistance of an electrical appliance is merely the rati current which that pressure can force through it. As a

$$\text{Resistance} = \frac{\text{pressure}}{\text{current}}$$

When the pressure is measured in volts and the cu sistance is then in ohms. Thus

$$\text{Ohms} = \frac{\text{volts}}{\text{amperes}}$$

Thus, since a pressure of 110 volts forces ½ ampere candescent lamp, the resistance of the lamp is 110/½ =

Ohm's Law. This relation between pressure, curren OHM'S LAW. It is written in symbols in the three form

$$R = E/I$$
$$E = IR$$
$$I = E/R$$

where

R = resistance in ohms;
E = pressure in volts;
I = current in amperes.

Example. An electric flat-iron has a resistance of will flow through it when it is put across a 110-volt c

$$I = E/R = 110/35 = 3.14 \text{ amper}$$

copper wire 1/10 in in diameter has a resistance of only 1/1000
er foot. Accordingly, because of their low resistance, copper wires
used to carry electric currents, and because of its high resistance,
nerally used as a covering of the copper wires to prevent leakage
e to another. Wire, approved by the National Board of Fire Under-
installed according to their rules, will have the proper insulating
each installation.

Watts. The flow of an electric current has been likened to the flow
ough a pipe. A current of water is measured by the number of
ounds, flowing per minute; a current of electricity is measured by
of amperes. The power required to keep a current of water flowing
ct of the current in POUNDS PER MINUTE by the head, or pressure,
his gives the power in FOOT-POUNDS PER MINUTE. To reduce to
it is necessary merely to divide by 33 000. Thus

$$\frac{\text{(pounds per minute)} \times \text{(feet)}}{33\,000} = \text{horse-power}$$

the same way, the POWER required to keep a current of electricity
e product of the current in AMPERES by the pressure in VOLTS.
e power in WATTS.

$$\text{Watts} = \text{amperes} \times \text{volts}$$

WATT is merely a unit of power, and denotes the power used when
ses one ampere of current to flow. The watts consumed when any
t flows under any pressure can always be found by multiplying the
mperes by the pressure in volts. Thus, if an incandescent lamp
pere when burning on a 110-volt line, the power consumed equals

$$0.5 \times 110 = 55 \text{ watts}$$

$$\text{Power} = \text{current} \times \text{pressure}$$

$$\text{Watts} = \text{amperes} \times \text{volts}$$

What power is consumed by a motor which runs on a 220-volt
takes 4 amperes?

$$\text{Watts} = \text{amperes} \times \text{volts} = 4 \times 220$$
$$\text{Power} = 880 \text{ watts}$$

ent lamps are rated as to the voltage of the line on which they can
as to the amount of electric power it takes to keep them glowing.
bon-filament lamp may be rated as a 110-volt, 50-watt lamp. A
p may be rated as a 110-volt, 25-watt lamp. This means that both
tended to run on a 110-volt circuit, but that it takes twice as much
p the carbon-filament lamp glowing as it does to keep the tungsten-
g.

er-Equation. The above relation between volts, amperes and
lly expressed in the form of an equation:

$$P = IE$$
$$I = P/E$$
$$E = P/I$$

ver in watts;
rent *in amperes;*
ure in volts.

Horse-Power-Hour. Kilowatt-Hour. When
power to run machinery, he has to pay not only acc
he uses but also according to the number of hours h
stance, he may use 40 horse-power for 1 hour and p
the rate of 3 cts for each horse-power-hour. If h
2 hours he would have to pay twice as much, becau
power twice as long. Another way of stating the
he used twice as many horse-power-hours. For in
40 × 1, or 40 horse-power-hours, and in the second
hours. In other words, he did twice as much work
did in the first, or received twice as much energy. T
then, is the HORSE-POWER-HOUR, and is the work do
power machine.

Example. How much work is done by a machine
is run for 8 hours?

1 h.p. in 1 hr does 1 h.p.-hr
15 h.p. in 1 hr does 15 h.p.-hr
15 h.p. in 8 hr does 8 × 15, or 120

That is

$$\text{Work} = \text{horse-power} \times \text{hours}$$

or

$$15 \times 8 = 120 \text{ h.p.-hr}$$

Similarly, electric power is sold by the KILOWATT-HO
or energy delivered in one hour by a 1-kilowatt mach
For lighting purposes electrical energy is usually so
kilowatt-hour. Thus at 12 cts per kw-hr the monthl
lamp on an average of 5 hours per day would be com
For 1 month of 30 days the lamp is burning

$$30 \times 5 = 150 \text{ hours}$$

To use a 40-watt lamp 150 hours consumes

$$40 \times 150 = 6\,000 \text{ watt-hours} = 6\,000/1\,000 =$$

zh to raise the temperature to a dangerous degree. On the other ıter current than allowed by this table is sent over the wire, the ill also rise, because the heating of a current is also directly pro- ıe SQUARE OF THE CURRENT. Thus, doubling the current which a carrying will quadruple the amount of heat which the wire must this Tables III and IV, see pages 1387 and 1388.

Circuit-Breakers. Use is made of the heating effect of a current ircuit against too much current, very much as a boiler is pro- fety-valve against too much pressure. A small piece of fusible ly a mixture of lead and bismuth, is inserted in the circuit in such the current which passes through the circuit must also pass through

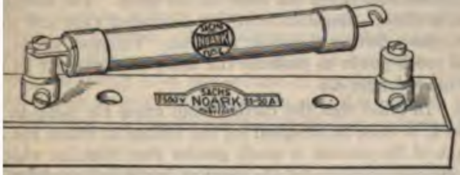


Fig. 2. Enclosed Fuse

ıetal. This device is called a FUSE. Any current which would be he circuit melts this fuse, opens the circuit at this point, and thus est of the circuit from the effects of the current. The cause of ent may be then removed and a new fuse inserted in place of the CUIT-BREAKERS are also used to protect a circuit against too much y are AUTOMATIC SWITCHES controlled by an electro-magnet and variety of styles. They operate upon the principle that when rrent passes through a coil of wire it makes a magnet of the coil.

LAMPS and ARC-LAMPS. The former are used principally for
although sometimes used for street-lighting, especially w
thickly shaded by trees. Arc-lamps are especially adapt
and for large interiors where they can be kept concealed o
the eye, as in railway-stations, stores, etc. An incande
monly made consists of a glass bulb containing a simple
conductor the ends of which are connected to the source of
When the current flows through the filament it heats it to
becomes incandescent; hence the name of the lamp. The
ment of finely-drawn tungsten represent the latest type
every way to those having a carbon filament. Tungsten-
one-third as much power to produce the same candle-po
and have a much longer life.

Voltages. In order that the current shall cause the la
CANDLE-POWER, it must be designed for the voltage at whi
If the voltage of the current is much greater than that f
designed it will quickly burn out the filament, while if the
is below that of the lamp, it will not give its rated candle-
lower reducing the candle-power about one-half. The vol
for tungsten-lamps is from 100 to 130. Tungsten-lamps a
ages of from 20 to 260. Two to four candle-power la
signs or decorative purposes, are made for from 10 to
steps, these lamps being commonly used in series, ten la
volt circuit. Two 5-watt lamps, 50 volts, are also ofte
100-volt circuit.

Candle-Power. Incandescent lamps of from 100 to 13
made 15, 20, 25, 40, 60, 100, 150, 250, 400 and 500 watts.
1 candle-power for every 1.1 watts. For the method of co
size and distribution of tungsten-lamps for illuminating a
1390 and 1391.

ed for stereopticon-lanterns may use as high as 25 amperes and
uld always be made in the wiring-plans for such a light for suffi-
wires to be installed to carry one and one-half times this current.

urrent Enclosed Arcs consume about 5 amperes at 80 volts, or
Arc-lamps generally require a resistance in series with the arc
gulate properly. This resistance is usually placed within the struc-
mp, and may be so adjusted that a single lamp can be made to
ny circuit from 100 to 130 volts.

lectric Machines. There are three classes of dynamo-electric

ATORS for generating an electric current.
s for converting electrical into mechanical energy.
ORMERS and ROTARY CONVERTERS.
ransformers for converting one voltage into a higher or lower
voltage. Converters and transformers belong to the same class.
otary converters for changing alternating currents to direct
currents or vice versa.

s either a motor or a generator. A MOTOR is the same machine as
ut with the nature of its operation reversed. GENERATORS are
al classes, namely, continuous-current and alternating-current
e latter are commonly called ALTERNATORS. Generators and
kinds vary in voltage, current and speed, according to the pur-
h they are designed. A TRANSFORMER consists essentially of two
ne coarse and one fine, wound upon an iron core. Its function
electrical energy from one voltage to another. If it reduces the
nown as a STEP-DOWN transformer, and if it raises it, it is known
transformer. A transformer has no moving parts and requires no

Currents Produced. There are two kinds of electrical currents
d for light and power in buildings, (1) DIRECT CURRENTS, and (2)
CURRENTS.
urrent is uniform in strength and direction, while an alternating
y rises from zero to a maximum, falls to zero, reverses its direction,
imum in the new direction and again returns to zero. A complete
hanges is called a CYCLE. The number of times the current goes
changes during each second is called the FREQUENCY of the current.
y commonly used for incandescent lighting is 60 cycles per
is, the current goes through the above changes in value 60 times
A frequency of 25 cycles is also in common use, especially for run-

Table I. Average Current Taken by Direct-Current Motors

Horse-power	Amperes on 110-volt line	Amperes on 220-volt line	Horse-power	Amperes on 110-volt line	Amperes on 220-volt line
¼	3	1.5	25	186	[illegible]
½	5.4	2.7	30	222	111
1	9	4.5	35	260	130
2	17	8.5	40	296	148
3	25	12.5	50	....	185
5	40	20	60	....	222
7½	58	29	75	....	275
10	76	38	85	....	312
15	114	57	100	....	366
20	150	75	....	....	

The current taken by single-phase alternating-current motors can be
by noting the current taken by a direct-current motor of the same size and
age, and dividing this current by the power-factor of the alternating-cur
motor. To find the current taken by each terminal of a three-wire, three-ph
alternating-current motor, divide the current taken by a single-phase altern
ing-current motor of the same size and voltage by 1.73.

Example. What current is taken by a 5-horse-power, alternating-curr
220-volt, induction-motor of 80% power-factor?

Solution. A 5-horse-power, direct-current, 220-volt motor takes 20 amper
A single-phase, 5-horse-power, 220-volt motor of 80% power-factor takes 20
= 25 amperes.

Electric-Lighting Systems Commonly Used for Supplying the Electrical Energy to Lamps

Direct-Current, Constant-Potential Systems. The systems most
in America are:

(1) Two-wire system largely used for incandescent lighting from sm
plants, as for a large office-building or factory. It is usually operated at
volts.

(2) Three-wire system used in small towns for the lighting of buildings fr
the public mains, usually operated at 220 volts. Also in large cities with und
ground conduit-system. See pages 1380 to 1382.

Five-wire and seven-wire systems with high voltage have been used
Europe, but very little in America.

Alternating-Current, Constant-Potential Systems. There are two sy
tems:

(1) Single-phase system. Current transmitted to building at from 1 000
2 000 volts and reduced to from 50 to 110 volts by a transformer. The te
phase is used in connection with alternating-current systems only in the se
of circuit. Thus a single-phase system means an alternating-current sys
sending out power from one circuit only of the generator. A three-phase sys
has three circuits.

(2) Three-phase system. Three or four wires are used. This system
most used for lighting from public plants, principally because it enables
lights and motors to be operated from the public dynamo and is the most econ
ical in wire. (See Table I.) Both of these systems are used for incandes
lighting and for power from central stations. For a comparison of a
wire direct current with a three-phase, three-wire alternating current

n alternating current may be changed to a direct current at a sub- rotary converter or by a mercury-arc rectifier. The latter is very ed in garages in order to convert an alternating current into a direct charging storage-

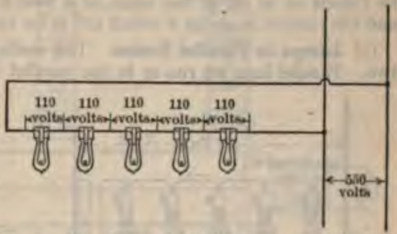


Fig. 3. Five Lamps in Series on a 550-Volt Line. Each Lamp has a Voltage of 110 Volts Across It

of Connecting here are three ways g lamps to the dis- res: (1) in series; allel; and (3) in s.

ps in Series. id to be connected n they are arranged he other, so that rent flows through . The most com- e of this system is the lighting of electric cars and the stations on ailway line. The voltage of such lines is usually 550 volts. Since incandescent lamp requires but 110 volts, five of these are placed n Fig. 3. Each lamp now has a pressure of 110 volts across it, and the set of five lamps requires 550 volts across it, and so can be placed across the railway supply-wires. When lamps are arranged in series the total resistance of the circuit is the sum of the resistances of the several parts, and the voltage required to force the current through a number of lamps in series is the sum of the voltages required for the separate lamps. Thus the voltage required to supply the proper current for four 52-volt lamps is 4 × 52 = 208 volts. Arc-lamps for street-lighting are often connected in series, but incandescent lamps are very seldom connected in series except as described above or for decorative purposes or electric signs. Where lamps of low voltage, as in signs, etc., are used on 110-volt systems it is necessary to connect them in series. The underwriters do not approve connecting incandescent lamps in series. The series system requires that the same current flow through each lamp, and if one lamp burns out the circuit is broken and all of the lamps will go out, unless some provision is made for maintaining the circuit around the dead lamps.

Lamps . Each he Full- e of 110 s It

(2) **Lamps in Parallel.** This is the common method of connecting incandescent lamps. It is illustrated in Fig. 4. With this system the pressure in each lamp is the same as in the distributing lines, and any lamp may be turned on or off without affecting the other lamps. For this system the PRESSURE or voltage must be kept constant, while the current or quantity of electricity flow- lines will depend upon the number of lamps that are burning. welve 16-candle-power lamps of 110 voltage on a parallel circuit, equiring 0.51 ampere when all the lamps are burning, a current of , *or 673.2* watts, will be required. With but one lamp burning,*

---

* *Watts being equal to amperes times voltage.*

a current of only 0.51 amper
same for one lamp as for the t
STANT-POTENTIAL system is re
be turned on or off at the la
and the contact made by a sw

(3) **Lamps in Parallel Se**
two. Parallel lines are run as

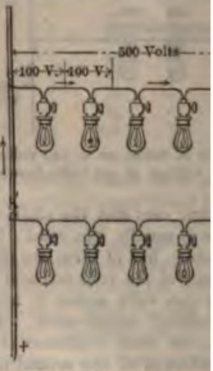


Fig. 5. Lamps in Parall

are connected in series betwee
necting lamps is used principa
on a power system. Fig. 5 sh
system and Fig. 6 a series of
lamps. Any number of series

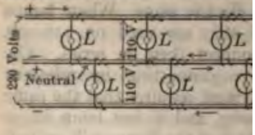


Fig. 7. The Three-wire Edis
Between Outside Wires; Only
Outside Wire and Neutral Wire

the mains. There are a nun
connection may be used.

The Edison Three-Wire
two-wire system of distribution
sized office-buildings, apartm
for motors is to be taken from
the power is not too great a po
wire system may also be used
cumstances be run for the r
the current-demand on the

the lamps, if on the same mains with the motors. Where com-
lines are required and the amount of current to be supplied is

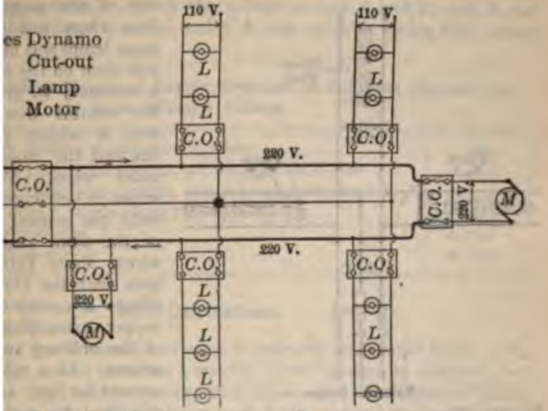


Fig. 8. Example of Three-wire System of Wiring

-WIRE SYSTEM is used. By this system two voltages or pressures
l, 110 and 220 volts being those generally adopted, the 110-volt
g the arc and
ghts and the
the motors.
ow the wires
connections
ssure between
wires is the
smitted from
usually 220
erior wiring.
n these two
pposite direc-
middle wire,
UTRAL WIRE,
of two cir-
ent from one
to flow in
nd that from
t in the oppo-
consequently
of the same
mperes, are
circuits they
other in the
nd there will
lowing in this

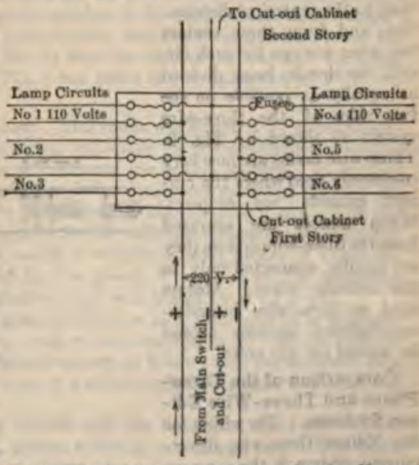


Fig. 9. The Wiring of a Cabinet. Showing How to Divide a Three-wire System into Six Two-wire Circuits, Three Circuits to Each Leg

current of 10 amperes flowing in one circuit and one of 6
ther circuit, the current flowing in the neutral wire will be 4

amperes. To obtain the greatest benefit from this system, it should alwa installed so that there will be nearly the same load or number of lamps on ea side of the neutral wire. Li then there will be times wh more lamps will be burning on o side than on the other, so tha is necessary to give some siz the neutral wire. The neut wire is seldom made less th one-half the cross-section of t outer wires. For distribut mains in buildings carrying la only, the neutral wire should of the SAME SIZE as the ou wires. From Table II it wil seen that the three-wire sys effects a considerable saving copper, amounting to fully of the ordinary two-wire 110- system. As a rule, in suppl current for light and power one plant, the main wires onl arranged on the three-wire sy and the distributing wires are on the two-wire system as in Fig. 8. When using the three-wire system lighting only, the three wires are usually run no farther within the bui than to the centers of distribution, and from these centers two wires are run for each circuit, the circuits being divided as equally as possible on the two sides of the three-wire system as shown by Fig. 9. Three-wire mains are now very commonly used where the current exceeds 100 amperes. When motors are operated from the three-wire system they are usually connected only to the outside wires. Motors used on three-wire incandescent-lighting systems should be wound for 220 volts.

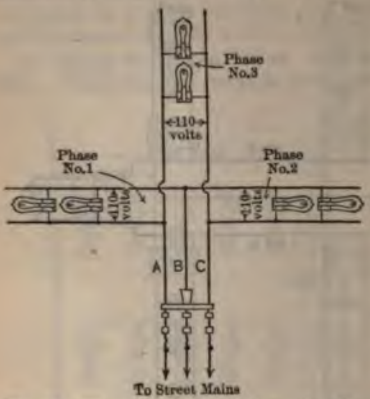


Fig. 10. Three-wire System, Alternating Current. Compare with Fig. 11

Comparison of the Three-Phase and Three-Wire Edison Systems. The wiring for the Edison three-wire direct-current system is the same as that for the three-wire, three-phase alternating-current system, the only difference being that the voltage BETWEEN ANY TWO WI a three-phase system is the same. Thus in Fig. 10 which represents a wire, three-phase system the voltage between the wires *A* and *B* (phase

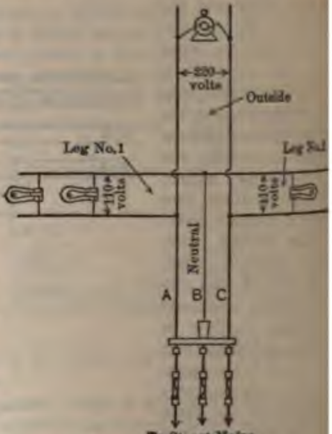


Fig. 11. Three-phase, Three-wire System, Current. Compare with Fig. 10

rrent, three-wire system, all wires of same size......	0.375
rrent, three-wire system, neutral, one-half size.......	0.313
ıg-current, single-phase two-wire system............	1.000
ıse three-wire..................................	0.750
ıse four-wire..................................	0.333

Wire-Calculations

ges. As the diameter of wires is ordinarily designated by the num-
gauge, and as there are a number of wire-gauges in common use,
ge of those used for copper wire is necessary. The Brown & Sharpe,
uge (see page 1388) is almost exclusively used in America in connec-
trical work, except where the size of the wire is designated in cir-
'he sizes of wire given by this gauge range from No. 0000 (0.46 in)
031 in), but No. 14 is the smallest size permitted for interior wiring.
vire has a diameter of about 1/10 in and its resistance per 1 000 ft
1 ohm. For any given number of this gauge a wire three numbers
ry nearly half the cross-section, and one three numbers lower has
ss-section; thus a No. 13 wire has very nearly one-half the cross-
No. 10 wire, and a No. 7 has twice the cross-section of a No. 10,
that of a No. 13.

lar-Mil Wire-Gauge. This gauge was designed by the engineer-
nt of the Edison Company especially for the designation of copper
rical work, and is now in general use in this country. In practice
ıuge is commonly used for designating wires up to No. 0 or No. 00,
above that size are designated by circular mils (c.m.). The size
ed is often determined in circular mils and designated by the corre-
& S. gauge-number, which is readily done by means of Table III,
Copper wire is sold by the pound if bare or of the numerous
f varieties, but rubber-covered wire is sold by the 1 000 ft.

To reduce circular mils t
ten places of decimals. Th

To obtain the sectional ar
Multiply together its dimen

Example. What is the s

Solution. ⅛ in = 0.125
250 × 1.273 = 39781.25 c.m
The weight of bare copp
weight of 1 000 ft of copper
× 2 000 = 6.054 lb. Table
bare copper wire from No.

Carrying Capacity of C
wire for interior wiring is p
pacity-limits given in the ta
to destroy the right to re
capacity of rubber-covere
Board of Fire Underwriters
pere-capacity assigned to
rubber insulation would d
as that allowed for weath
necessary a lower rate of he
No wire smaller than No.
No. 16 may be used for fl
11, 9 and 7 are not usuall
special order. Rubber-cov
ing-work and in damp pla
The latter wire may be
line-wires.

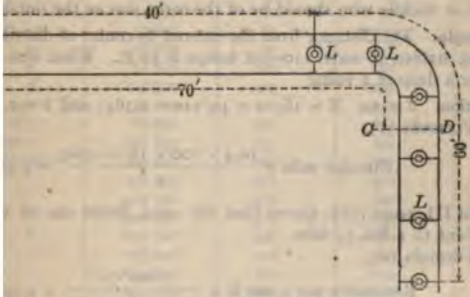
Drop of Potential. W
appreciable length the pres
so that if the current enters
circuit it will be somewhat l
the wire. This loss in volta
corresponds to LOSS OF HE
below that for which the la
it is very important that the
sufficient to affect the illun
wires has nothing to do wi
cause in the wires. Accord
of carrying the number of
and yet cause a drop of pote
will burn only at a dull red.
these mains and distributing
(1) That the wire is lar
to carry the current safely.
(2) That the potential dr
not be excessive. An exces
light and not enough copper
Where the current is sup
2% drop, but where the c
premises, a 3% or 5% dro
distances should be per

in percentage) = current in line × resistance of line, or drop
s × ohms.

it will be the drop in a circuit of No. 14 copper wire 280 ft
ne lamps, requiring 4.5 amperes?

ı Table V, page 1389, it is found that the resistance of No.
hms per 1 000 ft; hence for 280 ft it will be 2.527 × 0.280 =
drop in volts = 4.5 × 0.7075 = 3.1837 volts. The voltage for
ampere per lamp) will be about 110; consequently the per-
= 3.1837/110 = 2$\frac{9}{10}$%, nearly. A 2% drop on a pressure of
lts.

tribution. The meaning of this term may best be illustrated
Let Fig. 12 represent a circuit carrying six lamps, the first



The Point *D* is the Center of the Lamp-distribution

om the cut-out, or source of supply. The whole of the current
ted through this 40 ft, but from that point it will gradually
verage current will only extend to the point *CD*, halfway be-
e lamps. Or, in other words, the center of distribution is
center of gravity of the lamps on the circuit. The center of
nines the length of the line in the rules for finding the neces-

enters are the points in a building where the cut-out cabinets

or source of current to distributing center for main lin
in amperes PER LAMP. N = number of lamps sup
Both formulas apply to any voltage and to any two-w
formulas for the ordinary three-wire system, let
lamps on ONE SIDE of the neutral wire and DOUBLE
neutral or middle wire should be of the same size as

Example. The distance from the cut-out to center
carrying sixteen 40-watt, 110-volt lamps is 50 ft. W
used for a drop of 2 volts?

Solution. $d = 50$; $N = 16$; $c = 40/110 = 0.364$; a
By Formula (1),

$$\text{Circular mils} = \frac{10.4 \times 100 \times 16 \times 0.3}{2}$$

Table III, page 1387, shows that the next larger
equivalent to a No. 14 wire.
By Formula (2),

$$\text{Resistance per 1 000 ft} = \frac{1\,000 \times 2}{12 \times 0.364 \times 10}$$

which we see from Table IV, page 1388, is about the
but as No. 14 is the smallest wire permitted that size

Example. The distance from the entrance cut-out
building, to the main distributing center of a building
ber of 16-candle-power, 110-volt carbon-lamps sup
the size of the mains that should be used on the tw
of 2 volts? (A 16-candle-power 110-volt carbon l
0.51 ampere.)

Solution. $d = 100$; $N = 90$; $c = 0.51$; $v = 2$
By Formula (1),

$$\text{Circular mils} = \frac{10.4 \times 200 \times 90 \times 0.5}{2}$$

Table III. Carrying Capacity of Wires and Cables

FOR INTERIOR CONDUCTORS, ALL VOLTAGES

From the National Electrical Code

No. of wire, B. & S. gauge	Circular mils	Capacity in amperes	
		Rubber-covered	Weather-proof
18	1 624	3	5
16	2 583	6	10
14	4 107	15	20
12	6 530	20	25
10	10 380	25	30
8	16 510	35	50
6	26 250	50	70
5	33 100	55	80
4	41 740	70	90
3	52 630	80	100
2	66 370	90	125
1	83 690	100	150
0	105 500	125	200
00	133 100	150	225
000	167 800	175	275
0000	211 600	225	325
Cables	200 000	200	300
"	300 000	275	400
"	400 000	325	500
"	500 000	400	600
"	600 000	450	680
"	700 000	500	760
"	800 000	550	840
"	900 000	600	920
"	1 000 000	650	1 000
"	1 100 000	690	1 080
"	1 200 000	730	1 150
"	1 300 000	770	1 220
"	1 400 000	810	1 290
"	1 500 000	850	1 360
"	1 600 000	890	1 430
"	1 700 000	930	1 490
"	1 800 000	970	1 550
"	1 900 000	1 010	1 610
"	2 000 000	1 050	1 670

nt of one ampere will supply two 16-candle-power carbon lamps.

n this case we use one-half of N, or 45, and $2\,v$ instead of v; then

$$\text{Circular mils} = \frac{10.4 \times 200 \times 45 \times 0.51}{4} = 11\,920$$

OURTH the section required for the two-wire system. The size ed is No. 8; a No. 9 would answer if it could be had. Comparing wire required with the two-wire system gives two No. 3 wires lb per 1 000 ft, and with the three-wire system three No. 8 wires lb; hence, the saving in cost is nearly 50% and if No. 9 wire were e saving would be 55%. With a drop of 3% (3.3 volts) the cir-

$$\text{uired for the three-wire system} = \frac{10.4 \times 200 \times 45 \times 0.51}{6.6} = 7\,230,$$

requiring No. 10 wires. The
$c = 45.9$, and in the three-wire
it is seen that the smallest size
is No. 8; consequently, No. 8
comply with the underwriters'
0.2 × 0.6285 (amperes × resist

For the three-wire system, t
proof wire permitted by Tabl
volts, or 3.8 volts on each si
on very short lines a 2% dro
by the underwriters, and this

Table IV. Dimensions,

Gauge-number, B. & S.	Diameter in mils	Area in cir. mils
0000	460	211 600
000	410	167 800
00	365	133 100
0	325	105 500
1	289	83 690
2	258	66 370
3	229	52 630
4	204	41 740
5	182	33 100
6	162	26 250
7	144	20 820
8	128	16 510
9	114	13 090
10	102	10 380
11	91	8 234
12	81	6 530
13	72	5 178
14	64	4 107
15	57	3 257
16	51	2 583
17	45	2 048
18	40	1 624

* Approximate weight of weat
here given.

To find the smallest size of
it is only necessary to comput
select the wire having a capac
amperes. Table VI shows at
110-volt carbon lamps permitt

Formulas (1) and (2), page
required current in amperes is
peres for N × c.

V. Maximum Length of Line for Given Number of Lamps that can be Used with a Two-Per-Cent Drop. Two-Wire System

ased on ½ ampere per carbon-lamp. One 32-candle-power carbon-lamp = two 16-candle-power carbon-lamps. Four 40-watt tungsten-lamps = three 16-candle-power carbon-lamps

Number of 16-candle-power, 110-volt carbon-lamps								
4	6	8	10	11	12	16	20	24
Maximum length of line, one side, in feet								
209	139	104	83	76	70	52	42	35
.....	221	166	133	120	110	83	66	55
.....	.....	264	211	192	176	132	105	88
.....	.....	.....	326	297	272	204	163	136
.....	.....	.....	.....	......	440	334	267	220
Number of 16-candle-power, 110-volt lamps								
30	36	40	50	60	70	80	90	100
Maximum length of line, one side, in feet								
44	37	.....	.....	......	......	......	......	......
70	58	52	42	......	......	......	......	......
109	91	81	65	54	37	40	......	......
178	148	133	107	89	76	66	59	53
225	187	168	135	112	96	84	75	67
.....	236	212	170	141	121	106	94	85
.....	.....	268	214	180	153	134	119	107
.....	.....	.....	270	225	193	169	150	135
.....	.....	.....	.....	285	243	213	190	170

ree-wire mains with 220 volts between outer wires and same number of lamps on e, length of wire may be increased four times.

VI. Maximum Carrying Capacity of Wires in Terms of 16-Candle-Power 110-Volt Lamps, However Short the Wires May Be

Based on ½ ampere per lamp
Four 40-watt tungsten-lamps = three 16-candle-power carbon-lamps

Number of lamps		No. of wire, B. & S. gauge	Number of lamps	
Rubber-covered	Weather-proof		Rubber-covered	Weather-proof
24	32	4	130	184
34	46	3	152	220
48	64	2	180	262
66	92	1	214	312
92	130	0	254	370
108	154	00	300	440

Example. What should be
requires 30 amperes at 220 vo
pole, the drop in volts not to

Solution. Using Formula (1

Circular mil

which requires a No. 5 wire.

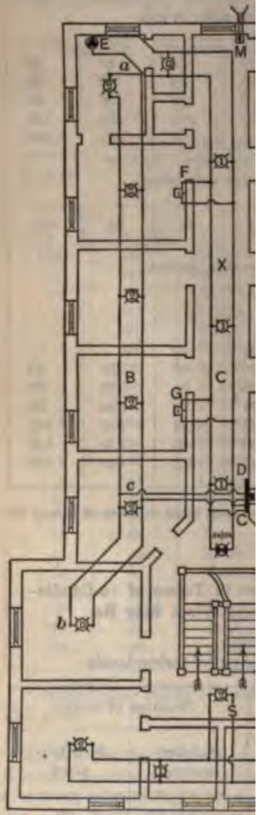


Fig. 13. Wiring-diagram for Sec
ing of Symbols, see p

in the specifications. Let us
lent to adding eight 40-watt
The meanings of the symbols

ircles denote the number of 16-candle-power carbon-lamps
same number of 25-watt or 40-watt tungsten-lamps may
out overloading the circuits. See pages 1398 and 1399
Symbols. The current to be obtained from the wires of
ompany, which carry a current at 220 volts between the
t 110 volts between either outside wire and the neutral
s for the building should enter through the alley-wall at
second
p in the
the wall
ock and
d be in
nd the
listribu-
be lo-
r of the
C, and
cabinet
m this
four cir-
, which
letters
ircuit *A*
n for a
near the
r rooms
in those
ights on
ntrolled
-sockets. The lights on circuits *B* and *D* are not switched,
head of stairs, which is controlled by a snap or push-button
first-class job all of the four circuits would be controlled
the cabinet, as shown in Fig. 14; but this is not absolutely

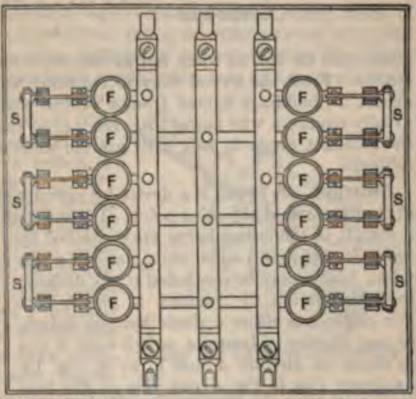


Fig. 14. Cabinet-wiring for Knife-switch Control

The center of distribution of circuits *A*, *C*, and *D* would be
marked *X* (Fig. 13). For circuit *B* take one-half the dis-
it the distance from *c* to the cabinet. In figuring the length
added for the drop from ceiling to the cabinet. Let us as-
lamps are to be used. In computing the current taken by
ys assumed that no smaller than a 40-watt tungsten is used.

ked-○- would probably be 25-watt lamps, but must be
according to the underwriters' rules. The number of 40-
th of wire for each circuit are as follows:

hts, 41 ft one way to center of distribution.
hts, 52 ft one way to center of distribution.
hts, 37 ft one way to center of distribution.
hts, 59 ft one way to center of distribution.
er of lamps, 47.

see that the maximum length of line one way for No. 14
carbon or sixteen 40-watt lamps is 70 ft. Consequently,
ts can be No. *14 wire, which is the smallest size* permitted.

Feed-Wires. These should be run on the three-wire system. 2 × 47 or 94 lamps in first and second stories and eight in basemen wires must be capable of supplying 102 lamps. Each 40-watt lamp 40/110 = 0.364 ampere. The distance from outside the building to cabinet is about 72 ft, allowing for three drops. Using Formu assuming that there will be fifty-one lamps on each side of the three- and doubling the drop in volts, gives

$$\text{Circular mils} = \frac{10.4 \times 144 \times 0.364 \times 51}{4} = 6\,960 \text{ c.m.}$$

which calls for No. 11 wire; but as this size is not carried in stock No. 10. From the second story to the third No. 12 wires could be almost all buildings lighted from a central station the lamp-circ usually require a wire larger than No. 14, so that about the only the architect needs to look after are the wires which run to the cabinets.

Switches. A switch is a device for opening or closing a circuit at the fixture or at any other point. In the better class of buildings if not all, of the ceiling-lights are controlled by switches placed at a convenient place on a side wall. Lights may be controlled at any distance from the fixture by running a switch-loop. For controlling either a single lamp or fixture, or any number of lamps, a switch-loop is run as shown on circuits *A* and *C*, as in Fig. 13. As shown also in Fig. 4, one side of the loop must be connected with one of the distributing wires and the other side to the lamp. When a number of to be controlled by one switch, as in the case of hall-lights, and in large rooms, such as churches, theaters, concert-halls, etc., circuit is usually run for those lamps, and a switch anywhere in distributing lines will turn on or off all of the lights on that li underwriters do not permit more than twelve 16-candle-power carbo 40-watt tungsten-lamps on one circuit, not more than these numbe can be controlled by one switch, except where the switch is pla mains. It is also practicable to control one lamp from two or t Thus by a duplex or three-point switch and proper wiring, a lamp ma or turned off from either the first or second story at will. By t three-point switches and one four-point switch a first-story hal

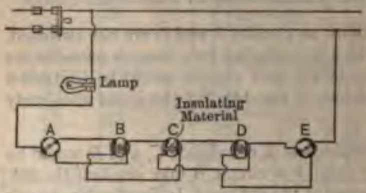


Fig. 15. The Lamp May Be Turned Off or On From Any of the Five Points, *A*, *B*, *C*, *D*, or *E*

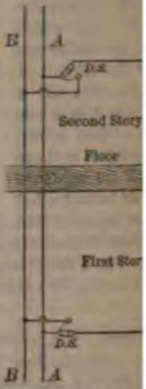


Fig. 16. The Lam Turned Off or On the First or Secon

utton switch is commonly used where a neat appearance is desir-
laces where this is of no
nap-switch is used, as it is
Vhere a circuit of twelve or
controlled by a switch, a
p.) knife-switch (Fig. 17)
sed, being generally placed
nife-switches should always
in wires. Snap and push-
are made both single and
SINGLE-POLE switch opens
the circuit and a DOUBLE-
oth sides. A double-pole
cessarily opens both sides.
on a three-wire system
poles. Double-pole snap and push-button switches are seldom
n twelve lamps. DUPLEX SWITCHES, sometimes called THREE-
, are usually of the snap or of the push-button type.

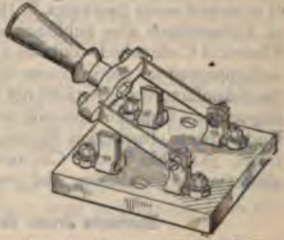


Fig. 17. Common Knife-switch

tems. As weather-proof or rubber-covered wire cannot be run
floors of brick, terra-cotta, or concrete without some protection
covering of the wires, it is necessary in such places to run the
r conduits, and in fire-proof buildings all of the lighting-wires
n in a system of conduits.

nduits. There are two kinds of interior conduits now in com-

d-Steel Pipe. The lining consists of a thin coat of enamel
impervious to water, sulphuric acid, acetic acid, hydrochloric
ate-of-soda solutions. For regular conduit systems only mild-
the same thickness as ordinary gas-piping is approved by the

code has also been made a part of the ordinances of r
It is revised every two years, in the odd-numbered year
of Underwriters also publishes, semi-annually, a SUPP
Electrical Code which contains a list of all articles th
and approved for use in connection with the code, tog
the manufacturers. Articles not included in this list w
inspectors. Copies of the code and supplement can be o
Underwriters' Inspection Bureau, or by writing to th
tories, 382 Ohio Street, Chicago, Ill. The following
almost every installation, and every architect should b

Extracts from the National Electrica

(1) All wire for concealed work must be of the best a
brands, as shown in List of Fittings. No wire smaller th
to be used. All wire run in conduits must have double-

(2) Where wires are concealed and run parallel to j
ported on porcelain knobs which hold the wires at lea
or surface wired over. Knobs must be SECURELY FASTEN
EVERY 4½ FT. Where wires are run through joists
with porcelain tubes the entire width of joists. All wir
so as to have all slack removed.

(3) In concealed work all wires MUST BE SEPARATED
AT LEAST 5 IN. Where wires run down partitions, espe
by 2 by 4-in studs, the wires must be so supported as
partition. If more than two wires are run down partiti
must be separated by at least 5 in.

(4) Where wires pass through floors they must be p
up to a point 5 ft above the floor with conduit or wit
always be a space of 1 in between the wires and the boxi

watts each.

t-outs must be placed in an ASBESTOS-LINED CABINET. The as- be at least ⅛ in in thickness and securely held in place by shellac Lumber of which cabinet is made must be at least ¾ in in thickness. t be furnished with snug-fitting door; door to be hung by strong o be furnished with a suitable catch.

outs to be approved must be of the plug or of the cartridge-type.

osed arc-lamps and incandescent lamps must not be placed on same s must be on separate circuits by themselves. Each arc-light must by an approved cut-out. The cut-outs are to be placed in an as- cabinet.

practice of using fused rosettes will not be approved, except in mills.

e wires run down the side wall they must be protected from me- ry.

utlets must be made to conform to Rule 24, National Electrical Code.

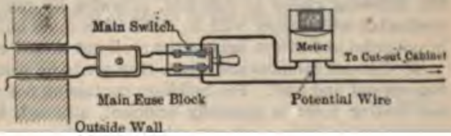
in series will not be approved.

of lamp-cord will not be approved. Lamp-cord is designed to be ps only. Ordinary insulated wire must be run to place desired.

ric heaters must be installed in accordance with Rule 25 a–f, National ode.

General Suggestions for Electric Work *

Principles and Recommendations. In all electric-work con- ever well insulated, should always be treated as bare, to the end conditions, kely to exist, ding or short , and so that from con- nductor, or nductor and



evenly as possible among tl
wiring avoided. The use
manently accessible is mos
method of accessible concea
itects are urged, when draw
the channeling and pocketi
in specifications for electric
the building is to be wired
gas-fixtures or gas-piping
Fig. 18 shows a common a
comply with Rule 7, page
near as possible to the outsi
the switch if desirable for a

Specifi

Specifications for Inter
(1) That the wiring shall
requirements of the Nation
and the rules of the local e
from the public mains.

(2) No electrical device o
by the Underwriters' Natio
the name or trade-mark of t
or other proper units mar
device is installed.

Requirements (1) and (2

(3) Contractor must obt
city inspector or from the i

(4) If the wires are to run
a conduit system is used, TH
ical work as far as possib
plastering is dry. All cond

(5) Size of Wires. The
to be less than No. 14 B. &
the following clause is suf
contractor will comply wit
in potential at farthest ligh

(6) Cut-out cabinets and
line cut-out and fuse. For
distributing point is gener
convenient to have a cut-o

(7) Number and kind o
plans, and the number of li
See pages 1398 and 1399 f
for controlling lights should

Approximate Cost of W
estimates of the cost of wiri
the number of outlets (not l
ber of pounds of wire requi
cut-out cabinets, etc., and

---

* Wiring specifications for b
pared by an expert.

t be made without plans and specifications. Again, wages and
ial vary to a considerable extent in different parts of the country,
nate that would be about right for one locality would not suffice
he following figures, however, will enable anyone to form an
ea of what any proposed wiring-job will cost.

f labor as not more than one-third the cost of the installation.

d-tube work in new houses of less than seventeen outlets or
nps, with no switches except main switch and a rough cut-out
asbestos, allow $1.50 per outlet.

ss of work, from 25 to 100 lamps, allow $1.75 to $2.00 per outlet.

bor involved in wiring old buildings will add from 30 to 50% to
es.

tch-loop with a single-pole snap-switch, add from $1.50 to $1.75.

tch-loop with single-pole push-button switch, add from $2.25 to

ıp controlled by duplex or three-point switches, add from $5 to

dwood cut-out cabinet with door and lock, add from $7 up ac-
ber of circuits and finish.

cabinets cost from $8.50 up.

osed wiring, as in factories, can usually be run for from $1.00
op, including rosettes, cord and sockets, the cost depending very
w closely the drops are spaced.

ations with iron-armored conduit will probably cost from $5 to
Large installations will cost somewhat less.

hting-plant of 200 lamps, wired on the concealed knob-and-tube
st from $1 250 to $1 500, and a similar plant with 600 lamps will
o to $3 000. These prices include engine, dynamo-switchboard,
and wiring, but no switches for controlling lamps.

ored conduit-system will add about $2.75 per outlet.

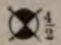
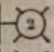
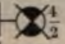
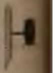
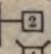
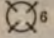
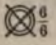
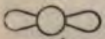
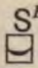
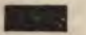
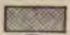
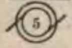
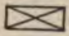
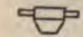
above estimates include the cost of fixtures except in the case of
.

d sockets cost about 90 cts per lamp. Single-lamp fixtures may
from $1.25 upwards; double-lamp fixtures from $2 upwards.
xtures cost about 25% more than straight electric fixtures.

rubber-covered wire varies from $8 to $60 per 1 000 ft according
weather-proof wire from 16 cts to 25 cts per pound.

Standard Wiring-Symbols Adopted by the National Contractors' Association and the American Institute of Architects

Copyrighted by the National Contractors' Association

	Ceiling-outlet; electric only. Numeral in center indicates number of standard 16-c.p. incandescent lamps.*	
	Ceiling-outlet; combination. $\frac{4}{2}$ indicates 4–16c.p. standard incandescent lamps and 2 gas-burners. If gas only.	
	Bracket-outlet; electric only. Numeral in center indicates number of standard 16-c.p. incandescent lamps.	
	Bracket-outlet; combination. $\frac{4}{2}$ indicates 4–16 c.p. standard incandescent lamps and 2 gas-burners. If gas only 	
	Wall or baseboard receptacle-outlet. Numeral in center indicates number of standard 16-c.p. incandescent lamps.	
	Floor-outlet. Numeral in center indicates number of Standard incandescent lamps.	
	Outlet for outdoor standard or pedestal, electric only. Numeral indicates number of standard 16-c.p. incandescent lamps.	
	Outlet for outdoor standard or pedestal; combination. $\frac{6}{6}$ indicates 6–16 c.p. standard incandescent lamps; 6 gas-burners.	
	Drop-cord outlet.	
	One-lamp outlet, for lamp-receptacle.	
	Arc-lamp outlet.	
	Special outlet for lighting, heating and power-current, as described in specifications.	
	Ceiling-fan outlet.	
S^1	S. P. switch-outlet.	Show as many symbols as there are switches. Or in case of a very large group of switches indicate number of switches by a Roman numeral, thus S′ XII, meaning 12 single-pole switches.
S^2	D.P. switch-outlets.	
S^3	3-way switch-outlet.	
S^4	4-way switch-outlet.	
S^D	Automatic door switch-outlet.	Describe type of switch in specifications, that is, flush or surface, push-button or snap
S^E	Electrolier switch-outlet.	
	Meter-outlet.	
	Distribution-panel.	
	Junction or pull-box.	
	Motor-outlet. Numeral in center indicates horse-power.	
	Motor-control outlet.	
	Transformer.	

* If tungsten-lamps are used instead of carbon-lamps, the figure in the circle will stand for the number of 25-watt tungsten-lamps, a 25-watt tungsten-lamp being the nearest in candle-power to a 16-candle-power carbon-lamp though consuming less than one-half the power. Since tungsten-lamps average about 1.1 watts to the candle-power many architects place in the circle the number of watts to be used. Dividing this

ring-Symbols Adopted by the National Contractors' Association ad the American Institute of Architects (Continued)

Main or feeder-run concealed under floor.

Main or feeder-run concealed under floor above.

Main or feeder-run exposed.

Branch circuit-run concealed under floor.

Branch circuit-run concealed under floor above.

Branch circuit-run exposed.

Pole-line.

Riser.

Telephone-outlet; private service.

Telephone-outlet; public service.

Bell-outlet.

Buzzer-outlet.

Push-button outlet. Numeral indicates number of pushes.

Annunciator. Numeral indicates number of points.

Speaking-tube.

Watchman-clock outlet.

Watchman-station outlet.

Master time-clock outlet.

Secondary time-clock outlet.

Door-opener.

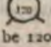
Special outlet for signal-systems, as described in specifications.

Battery-outlet.

Circuit for clock, telephone, bell or other service, run under floor, concealed. Kind of service wanted ascertained by symbol to which line connects.

Circuit for clock, telephone, bell or other service, run under floor above, concealed. Kind of service wanted ascertained by symbol to which line connects.

of wall-outlets (unless otherwise specified):
s.. 5 ft 6 in
.... 5 ft 0 in
.... 6 ft 0 in
.... 6 ft 3 in
es (unless otherwise specified)........................... 4 ft 0 in

candle-power per outlet. Thus  means enough tungsten-lamps this outlet to total 120 watts, three 40-watt lamps, or two 60-watt candle-power in any case would be 120/1.1 = 110 candle-power.

study of the problems involved, many of them have be
a wholly unnecessary mystery. Of none, perhaps, is
ARCHITECTURAL ACOUSTICS. The conditions surround
speech in an enclosed auditorium are complicated, it i
as will yield an exact solution in the light of adequ
reasonable, therefore, to include problems of architect
RATIONAL ENGINEERING PROBLEMS.

Character and Application of the Problem. T
tural acoustics is necessarily complex, and each room
which contribute to the result in a greater or less deg
stances. To take justly into account these varied c
the problem should be QUANTITATIVE, not merely QU
its highest usefulness and the dignity of an engineering
that its application can precede, not merely follow,
building.

Conditions and Factors of the Problem. In or
good in any auditorium it is necessary that the sound s
that the simultaneous components of a complex sou
proper relative intensities, and that the successive s
articulation, either of speech or of music, should be cle
each other and from extraneous noises. These three
are the entirely sufficient, conditions for good hearing.
lem involves three factors:

(1) Reverberation.
(2) Interference.
(3) Resonance.

As an engineering problem it involves the shape of t
sions, and the materials of which it is composed.

Rate of Absorption of Sound. Sound, being en
confined space, will continue until it is either transmitt
or is transformed into some other kind of energy, gene

was so small that a word spoken in an ordinary tone of voice was and a half seconds afterwards. During this time even a very ker would have uttered the twelve or fifteen succeeding syllables. ssive enunciations blended into a loud sound, through which and was necessary to hear and distinguish the orderly progression of cross the room this could not be done; even near the speaker e only with an effort wearisome in the extreme if long main-

flection, Reverberation and Echoes. With an audience filling onditions were not so bad, but still not tolerable. This may be e so chooses, as a process of MULTIPLE REFLECTION from walls, d from floor, first from one and then another, losing a little at until ultimately inaudible. This phenomenon will be called , including, as a special case, the ECHO. It must be observed, in general, reverberation results in a mass of sound filling the incapable of analysis into its distinct reflections. It is thus more gnize and impossible to locate. The term ECHO will be reserved lar case in which a short, sharp sound is distinctly repeated by r once from a single surface, or several times from two or more

cay of Sound. In the general case of reverberation we are with the RATE OF DECAY of the sound. In the special case of concerned not merely with its intensity, but with the interval between the initial sound and the moment it reaches the ob- room mentioned as the occasion of this investigation no discrete tly perceptible, and the case will serve excellently as an illustra- general type of reverberation.

Audibility of Residual Sound. After preliminary gropings, rature and then with several optical devices for measuring the nd, all established methods were abandoned. Instead, the RATE measured by measuring what was inversely proportional to it, audibility of the reverberation, or, as it will be called here, the UDIBILITY OF THE RESIDUAL SOUND. These experiments may be vantage here, for they will give more clearly than would abstract ea of the nature of reverberation.

oom and Nature of Furnishings. Broadly considered there y two, variables in a room, SHAPE (including size), and MATERIALS shings). In designing an auditorium an architect can give con- oth; in repair-work for bad acoustic conditions it is generally o change the shape, and only variations in materials and allowable. This was, therefore, the line of work in this

e Absorbing Power of Different Substances. It was evident gs being equal, the rate at which the reverberation would dis- portional to the rate at which the sound was absorbed. The fore, was to determine the RELATIVE ABSORBING POWER of various ith an organ-pipe as a constant source of sound, and a suitable recording, the duration of audibility of a sound after the source his room when empty was found to be 5.62 seconds. All the he seats in Sanders Theater, Boston, Mass., were then brought d in the lobby. On bringing into the lecture-room a number

time. Finally, when all the cushions from a th
persons were placed in the room, covering the seat
and the rear wall to the ceiling, the duration of
sound was 1.14 seconds. This experiment, requirin
work, having been completed, all the cushions we
was in readiness for the test of other absorbent
STANDARD OF COMPARISON had been established.
meters wide and 17 meters in total length, were
duration of audibility was then 4.51 seconds. Turni
been collected, it appeared that this amount of ch
meters of cushions from Sanders Theater. Oriental
Hindoostanee) were tested in a similar manner, a
canvas, and hair-felt. Similar experiments, but in
the absorbing power of a man and of a woman, alwa
ber of running meters of Sanders Theater cushions t
effect. This process of comparing two absorbents
for the other is laborious, and it is given here only t
development of a method. Without going into de
say that this method was so perfected as to give not
LUTE, COEFFICIENTS OF ABSORPTION.

Coefficients of Absorption. In this manner
OF ABSORPTION were determined for objects and
brought into and removed from the room, for soun
above middle C. In the following table the num
LUTE COEFFICIENTS OF ABSORPTION:

Oil-paintings, inclusive of frames. . . . .
Carpet-rugs. . . . .
Oriental rugs, extra heavy. . . . .
Cheese-cloth. . . . .
Cretonne cloth. . . . .
Shelia curtains. . . . .

e, per person	0.44
woman	0.54
man	0.48
h settees	0.039
h settees, per single seat	0.0077
h chairs, bent wood	0.0082
ered settees, hair and leather	1.10
ered settees, per single seat	0.28
ered chairs, similar in style	0.30
hions, per seat	0.21
elt cushions, per seat	0.20

f Absorption of Floors, Ceilings and Wall-Surfaces. Of ortance was the determination of the COEFFICIENT OF ABSORP- eilings, and he accom- called for a e extension dopted. If in a room he addition aterial are lting curve be a portion with dis- example of obtained in of the Fogg Cambridge, ed in the 1. If now is curve is the axes of coordinates are the asymptotes of the rectangular 2), the displacement of the origin measures the initial absorbing m, its floors, walls and ceilings. Such experiments were carried umber of rooms in which the different component materials different degrees, and an elimination between these different ve the following COEFFICIENT OF ABSORPTION for different

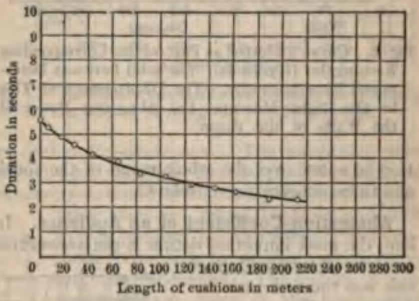


Fig. 1. Curve Showing the Relation of Duration of Residual Sound to Added Absorbing Material

ndow	1.000
sheathing, hard pine	0.061
on wooden lath	0.034
on wire lath	0.033
ingle thickness	0.027
on tile	0.025
t in Portland cement	0.025

the Reverberation for Any Room. If the experiments in plotted in a single diagram, the result is a family of HYPERBOLAS a very interesting relationship to the volumes of the rooms. hese hyperbolas the parameter, which equals the product of the coordinates, is determined, it will be found to be linearly proportional volume of the room. These results are plotted in Fig. 4, showing ho the proportionality is even over a very great range in volume. We ha at hand a ready of calculating the REV ATION for any roo volume and the mate which it is compose known. The first fi of the investigatio devoted to violin C, t octave above middle ing a VIBRATION-FRE of 512 vibrations pe This pitch was ch cause, in the art of t it was regarded at t as the characteris determining the of articulate spe planning of Symph Boston, Mass., extension of this tion to notes over the whole range of the musical scale, three oct and three octaves above violin-C.

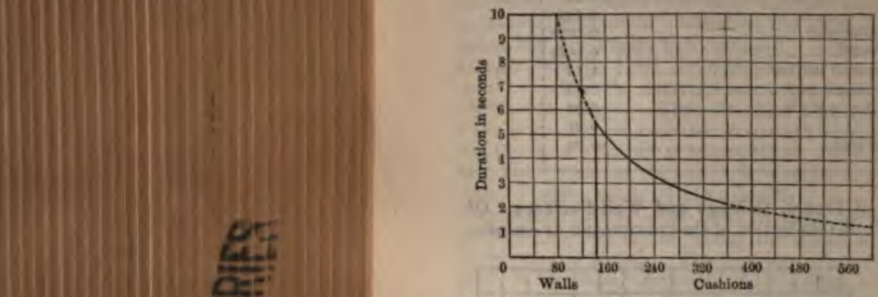


Fig. 2. Curve 5 Plotted as Part of its Corresponding Rectangular Hyperbola. The Solid Part was Determined Experimentally. The Displacement of This to the Right Measures the Absorbing Power of the Walls of the Room

Absorption-Coefficient of an Audience. In the very nature of lem, the most important datum is the ABSORPTION-COEFFICIENT of a and the determination of this was the first task undertaken. By means of a lecture on one of the recent developments of physics, wireless telegraphy, an audience was thus drawn together and at the end of the lecture requested to remain for the experiment. In this attempt the effort was made to determine the coefficients for the five octaves from C_2128 to C_62048, including notes E and G in each octave. For several reasons the experiment was not a success. A threatening thunderstorm made the audience a small one, and the sultriness of the atmosphere made ope necessary; while the attempt to cover so many notes, thirteen in al the experiment beyond the endurance of the audience. While this failed, another, the following summer, was more successful. In the had elapsed the necessity of carrying the investigation further tha intended became evident, and now the experiment was carried fr

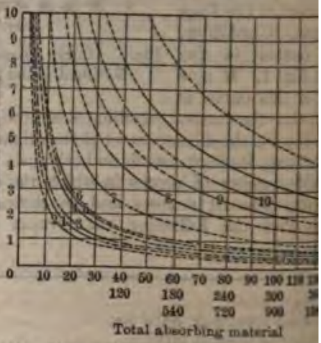


Fig. 3. Curves Entered as Parts of their Co Rectangular Hyperbolas. Three Scales a for the Volumes, by Groups, 1–7, 8–21,

included only the C notes, seven notes in all. Moreover, bearing
xperiences of the previous summer, it was recognized that even
would come dangerously near overtaxing the patience of the
Inasmuch as
ENT OF AB-
r C_4512 had
determined
re, in the in-
nentioned, the
this note was
nined. The
vas therefore
for the lower
upper three
seven. The
the night of
nt, was much
at which came
summer, the
more com-
and it was

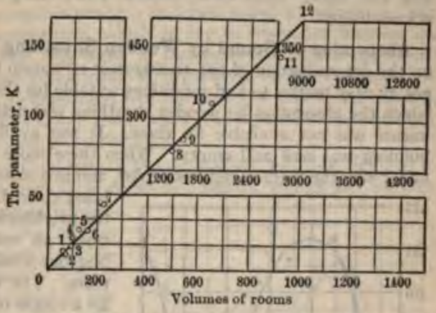
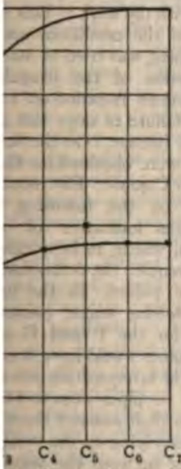


Fig. 4. The Parameters k, Plotted Against the Volumes of the Rooms, Showing the Two Proportional

lose the windows during the experiment. The conditions were
tisfactory. In order to get as much data as possible, and in as
there were nine observers stationed at different points in the room.

These observers, whose kindness and skill it is a pleasure to acknowledge, had prepared themselves, by previous practice, for this one experiment. The results of the experiment are shown on the lower curve in Fig. 5. This curve gives the COEFFICIENT OF ABSORPTION PER PERSON. It is to be observed that one of the points falls clearly off the smooth curve drawn through the other points. The observations on which this point is based were, however, much disturbed by a street-car passing not far from the building, and the departure of this observation from the curve does not indicate a real departure in the coefficient, nor should it cast much doubt on the rest of the work, in view of the circumstances under which it was secured. Counteracting the, perhaps, bad impression which this point may give, it is considerable satisfaction to note how accurately the point for C_4512, determined six years before by a different set of observers, falls on the smooth curve through the remaining points. The upper curve represents the absorbing power of an audience per square meter, as ordinarily



bing Power of an Audi-
Different Notes

vertical ordinates are expressed in terms of total absorption
neter of surface. For the upper curve the ordinates are thus the
ficients of absorption. The *several notes are at octave-intervals*

as follows: $C_1$64, $C_2$128, C_3 (middle C) 256, $C_4$512, $C_5$1024, $C_6$204
In the audience on which these observations were taken there were
and 105 men. The courtesy of the audience in remaining for the
and the really remarkable silence which they maintained are
acknowledged.

Absorption of Sound by Wooden Sheathing. The next exper
on the determination of the ABSORPTION OF SOUND by wooden sheath
not an easy matter to find conditions suitable for this experiment. T
which the absorption by wooden sheathing was determined in the ea
iments was not available for these. It was available then only b
building was new and empty. When these more elaborate experi
under way the room became occupied, and in a manner that did not
being cleared. Quite a little
the neighborhood of Boston
cover an entirely suitable
best one available adjoined a
room. The night-lunch was
for a couple of nights, and the
was tried. The work of both
much disturbed. The traffi
building did not stop until
o'clock, and began again at
interest of those passing on
out the night, and the necessit
explanations to the police,
fered with the work. This d
ment of the conditions und
experiment was tried is mad
explanation of the irregul
observations recorded on th
of the failure to carry this p
of work further. On the firs
points were obtained for th
$C_1$64 to $C_7$4096. The reduc
results on the following
variations indicative of
minima, which, to be accura
would require the determina
mediate points. In the ex
the following night, points
mined for the E and G n
octave between $C_2$128 and $C_6$2048. Other points would have been
but time did not permit. It is obvious that the intermediate points
and in the higher octave were desirable, but no pipes were to be
short notice for this part of the range, and in their absence the da
be obtained. In the diagram, Fig. 6, the points lying on the vertic
determined the first night. The points lying between the vertic
determined the second night. The sheathing of the room is of No
pine, 2 centimeters thick. The absorption is here due almost wholly
of the sheathing as a whole. It is not possible now to learn as mu
to the framing and arrangement of the studding in the particular
as is desirable. The accuracy with which these points fall on a smo
perhaps, all that could be expected in view of the difficulty under wh

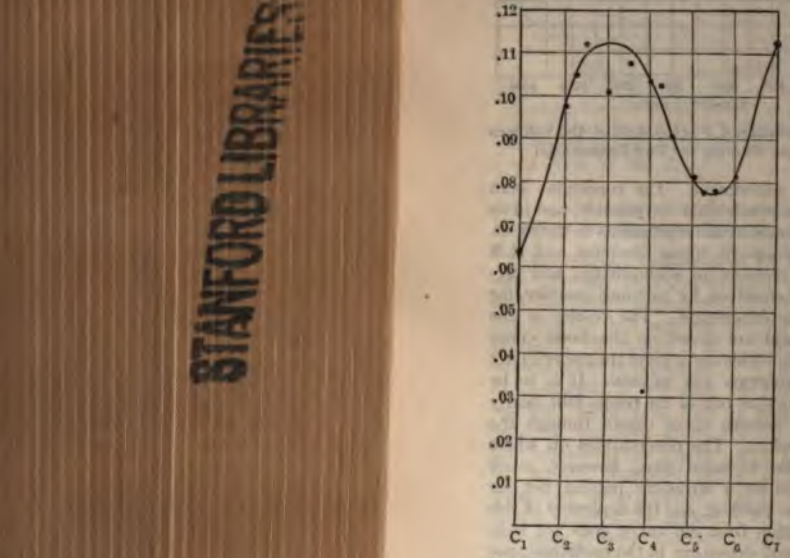


Fig. 6. Absorbing Power of Wooden Sheathing

ere conducted and the limited time available. One point in par- ar off from this curve, the point for $C_1$256, by an amount which least, serious, and which can be justified only by the conditions the work was done. The general trend of the curve seems, how- hed beyond reasonable doubt. It is interesting to note that there of MAXIMUM ABSORPTION, which is due to resonance between the e sound, and that this point of maximum absorption lies in the hough not in the lowest part, of the RANGE OF PITCH tested. It een interesting to determine, had the time and facilities permitted, he curve beyond $C_7$4096, and to see if it rises indefinitely, or shows, likely, a succession of maxima.

n of Sound by Cushions. The experiment was then directed to ation of the ABSORPTION OF SOUND by cushions, and for this purpose ade to the constant-temper- Working in the manner the earlier papers for sub- could be carried in and out e curves represented in Fig. ined. Curve 1 shows the OEFFICIENT for the Sanders ions, with which the whole was begun ten years ago e cushions were of a partic- rade of packing, a sort of vegetable fiber. They were canvas ticking, and that, in very thin, cloth covering. or cushions borrowed from rooks House. They were of filled with long, curly hair, with canvas ticking, which overed by a long-nap plush. r the cushion of Appleton covered with a leatherette, sharper maximum and a minution in absorption for equencies, as would be ex- such conditions. Curve 4 e most interesting, because for more standard commercial con- rily used in churches. This curve is for the elastic-felt cushions of elastic cotton covered with ticking and short-nap plush. The er is per square meter of surface. It is to be observed that all all off for the higher frequencies, all show a maximum located tave, and three of the curves show a curious hump in the second break in the curve is a genuine phenomenon, as it was tested ne. It is perhaps due to a SECONDARY RESONANCE, and it is to that it is the more pronounced in those curves that have the ance in their principal maxima.

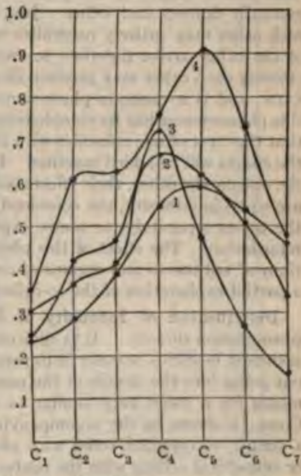


Fig. 7. Absorbing Power of Cushions

Interference of Sound-Waves. In both articulate speech and source of sound is rapidly and, in general, abruptly changing in and loudness. In music one PITCH is held during the length of rticulate speech the unit or ELEMENT OF CONSTANCY is the syllable. ech it is even less *than the length of a syllable, for the open-vowel*

If there are a number of such trains of waves thus
be that of MULTIPLE ECHOES. On the other hand,
traveled so nearly equal paths that they overlap,
difference in length of the paths which they have
mutually destroy each other. Just as two equal tr
each other may entirely neutralize each other if the
of the other arrive together, so two sounds, comin
crossing each other may produce silence. This phe
ENCE, and is a common phenomenon in all types
this phenomenon has its complement. If the two tr
that the crest of one coincides with the crest of the o
the effects will be added together. If the two sound-
the one on the other, their effects will also be added.
are equal in intensity, the combined intensity will b
the trains separately, as above explained, or zero,
retardation. The effect of this phenomenon is to p
rium of LOUDNESS and regions of comparative or e
a partial explanation of the so-called DEAF REGIONS

Distribution of Intensity of Sound. It is
phenomenon directly. It is difficult, however, to
nomenon in such a manner as to permit of an accura
out going into the details of the method employed,
ments for a room very similar to the Congregati
Conn., is shown in the accompanying chart. The
a simple, rectangular room with plain side walls a
or cylindrical ceiling with the center of curvature a
is clearly represented in Fig. 8, in which the INTENS
indicated by contour-lines in the manner employed
detic survey-maps. The phenomenon indicated
ephemeral, but was constant so long as the source
peated itself with almost perfect accuracy day after
enon one which could be observed merely instru
moving about in the room it was quite as striking

nd under these conditions is called the INTERFERENCE-SYSTEM,
ir note, of the room or space in question. If the source of sound
ped, it requires some time for the sound in the room to be ab-
rolongation of sound after the source has ceased is called REVER-
he source of sound, instead of being maintained, is short and
as a discrete wave or group of waves about the room, reflected

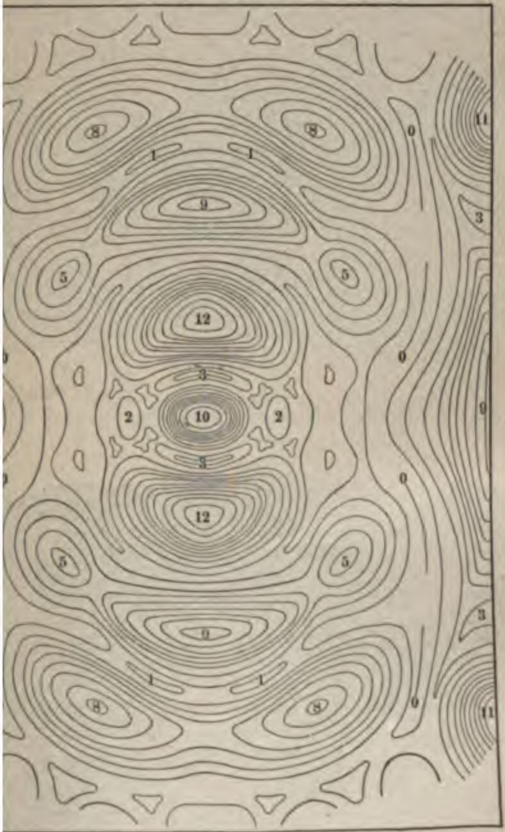


Fig. 8. Distribution of Intensity of Sound

ll, producing echoes. In the Greek theater there was ordinarily
doubling the case-ending," while in the modern auditorium there
rally arriving at a less interval of time after the direct sound, and
istinguishable, but stronger and therefore more disturbing.

ing Air-Disturbances. The formation and the propagation of
admirably studied by an adaptation of the so-called SCHLIEREN-

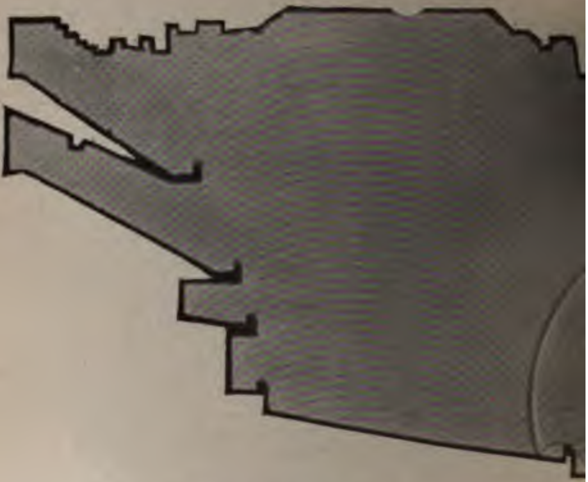


Fig. 9. Photograph of Sound-wave. Vertical Section

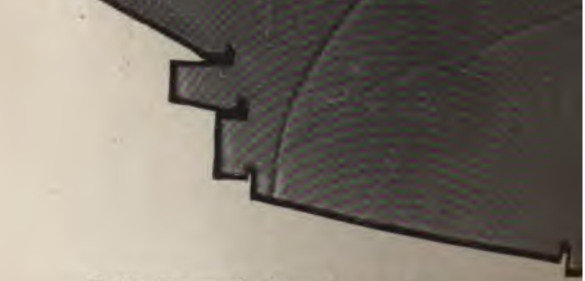
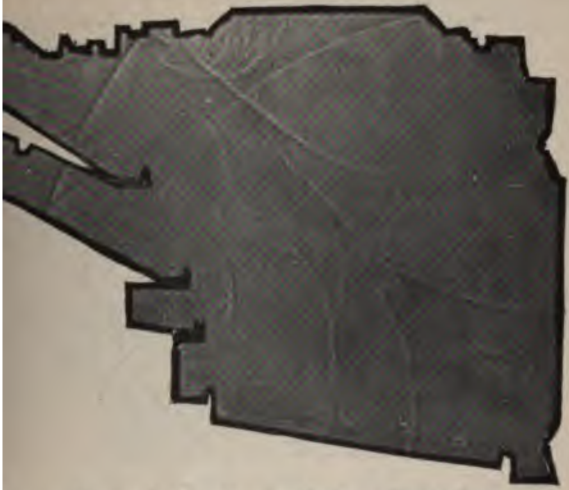


Fig. 10. Photograph of Sound-wave. Vertical Section



ig. 11. Photograph of Sound-wave and Echoes. Vertical Section

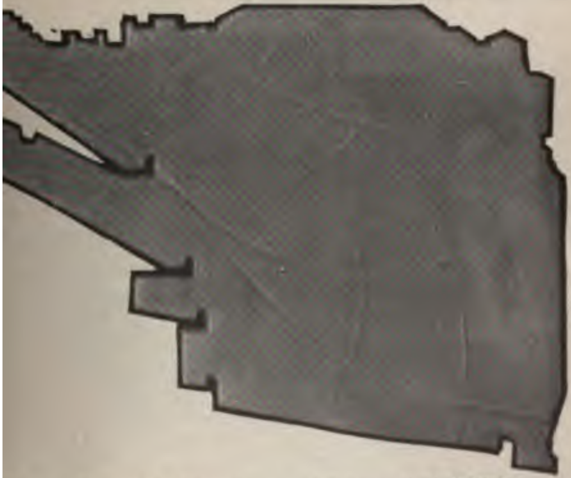


Fig. 12. Photograph of Sound-wave and Echoes. Vertical Section

METHODE device for photographing air-disturbances. It is sufficient here that the adaptation of this method to the problem in hand consists in th struction of a MODEL in proper scale, of the auditorium to be studied and a

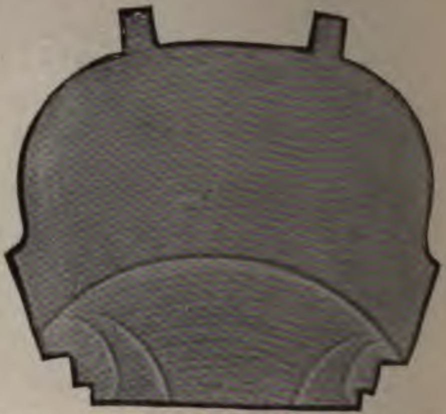


Fig. 13. Photograph of Sound-wave and Echoes. Horizontal Secti

tigation of the propagation through it of a proportionally scaled sound-wa examine the formation of echoes in a vertical section, the sides of a m taken off and, as the sound is passing through it, it is illuminated instant

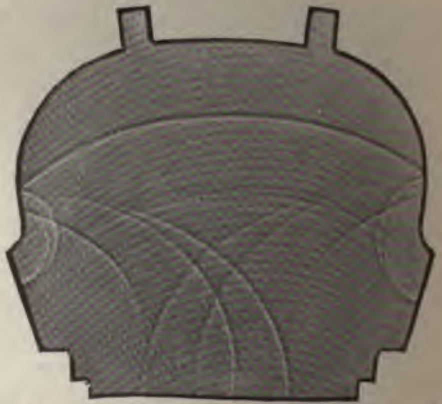
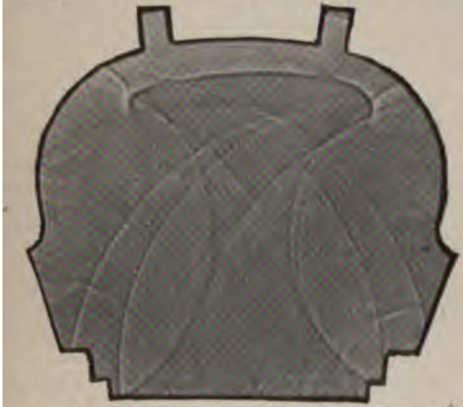


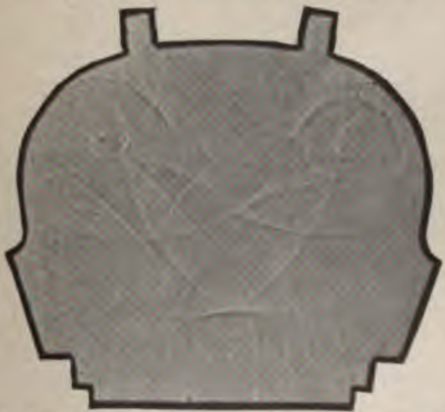
Fig. 14. Photograph of Sound-wave and Echoes. Horizontal Section

by the light from a very fine and somewhat distant electric spark. In the panying illustrations, reduced from the photographs, the silhouettes show the shadows cast by the model, and all within are direct photographs of the and its echoes. Figs. 9 to 12 show the sound and its echoes at ages in their propagation through the room, the particular part of ium under investigation being the New Theater in New York City. It



. 15. Photograph of Sound-wave and Echoes. Horizontal Section

ult to identify the MASTER-WAVE and the various ECHOES which it or, knowing the velocity of sound, to compute the interval at which heard. To show the generation of echoes and their propagation in



. 16. Photograph of Sound-wave and Echoes. Horizontal Section

l plane, the ceiling and floor of the model are removed and the photo-n in a vertical direction. The photographs shown in Figs. 13 to 16 choes produced in the horizontal plane passing through the marble front of the box.

Solution of Problems Possible in Advance of Construction. Whi these several factors, REVERBERATION, INTERFERENCE and ECHO, in an au torium at all complicated are themselves complicated, nevertheless they a capable of an exact solution, or, at least, of a solution as accurate as are th architect's plans in actual construction; and it is entirely possible to calcul in advance of construction whether or not an auditorium will be good, and i not, to determine the factors contributing to its poor acoustics and a method fo its correction.

SPECIFIC GRAVITY

The Specific Gravity of a substance is the number which expresses the that the weight of a given volume of the substance bears to the weight of th same volume of distilled water at a temperature of 62° F.; or, the specific gra of a body is equal to its weight divided by the weight of an equal volume of wa The specific gravity of a substance, multiplied by the weight of a cubic foot water, will give the weight of a cubic foot of the given substance. The weig of a cubic foot of water, at 62° F. and at the sea-level, is about 62.355 lb.* Th specific gravity of a solid substance may be determined by first weighing a po tion of it in air and then in water and dividing the weight in air by the loss of th weight in water; the quotient is the specific gravity required.

Example. A piece of granite weighs 5.32 lb in air; when immersed in wa it weighs 3.32 lb.

Solution. Weight in air (5.32 lb) divided by loss of weight in water (2 lb) 2.66, the specific gravity.

$$2.66 \times 62.355 \text{ lb} = 165.84 \text{ lb} = \text{weight per cubic foot}$$

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* The textbooks differ slightly in regard to this value.

ravities and Weights per Cubic Foot of Various Substances *

or specific gravities is pure water at 62° F., eter 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
2.5 to 2.8	2.6	162.1
eric at 60° F., under pressure of one atmos- .7 lb per sq in, weight 1/815 the weight of water	0.00123	0.0767
arbonate 2.61 to 2.76	2.68	167.1
olute, at 32° F.	0.794	49.5
er cent	0.934	58.24
er cent	0.815	50.82
nmercial	0.833	51.95
0.42 to 1.01	0.55	34.3
	1.72	107.4
hammered	2.75	171.7
drawn	2.68	167.1
sheet	2.67	166.5
pure	2.67	166.5
cast	2.56	160.0
13.7 to 14.1	13.92	868.0
	1.08	67.4
	0.87	54.3
o° F.	0.894	55.81
ast	6.70	418.0
native	6.67	416.0
dry † 0.66 to 1.25	0.75	46.8
5.7 to 5.8	5.73	357.3
2.1 to 3.0	2.81	175.0
athing-paper	1.20	75.0
an white, dry †	0.61	38.0
coal, solidly packed	0.70	40 to 45
street-paving	1.60	100.0
1.11 to 1.23	1.15	69 to 75
k, gravel	1.79	111.6
y †	0.36	22.5
	3.88	242.0
	4.45	277.5
ap-rock, average	2.96	184.6
y, N. J.	3.00	187.1
inn.	2.95	184.0
and, N. Y.	2.86	178.3
06.5 to 1.12	0.74	46.0
	0.95	59.0
	0.69	43.0
	1.04	64.9
0.52 to 1.08	0.65	40.6
st 9.76 to 9.90	9.82	612.3
° F.	1.06	66.2
1.8 to 2.0	1.90	118.6
1.7 to 1.8	1.75	109.2
rench, dry †	1.33	83.0

s given in this table for specific gravities and for weights per cubic foot are s. In the computations and compilations of these tables the Editor ebted to Mr. T. Z. Talley for valuable assistance.

"dry" in this connection indicates that the wood contains not more than re. Green timbers usually weigh from one-fifth to nearly one-half more inary building-timbers, tolerably *seasoned, one-sixth more.*

Specific Gravities and Weights per Cubic Foot of Various Subst
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1
Boxwood, Dutch, dry †	1.035
Boxwood, Brazilian, dry †	
Brass (copper and zinc), cast 7.8 to 9	8.45
Brass, rolled	8.56
Brass, sheet	8.24
Brass, wire	8.69
Bricks, building	....
Bricks, common	1.922
Bricks, light, inferior	1.442
Bricks, lime-sand	2.163
Bricks, Magnesia	2.643
Bricks, pressed	2.163
Bricks, pressed, hard	2.403
Bricks, soft	1.602
Bricks, fire	2.403
Bricks, paving	
Brickwork, pressed brick, fine joints	2.24
Brickwork, medium quality	2.00
Brickwork, coarse, inferior, soft	1.60
Brickwork, at 125 lb per cu ft, 1 cu yd equals 1.507 tons and 17.92 cu ft equal 1 ton	....
Bromine	3.19
Bronze, coin	8.66
Bronze, gun-metal	8.60
Bronze, ordinary	8.40
Bronze, aluminum	7.70
Butter	0.86
Butternut-tree, dry †	0.38
Cadmium 8.6 to 8.7	8.65
Calcite 2.6 to 2.8	2.70
Calcium	1.58
Camphor, dry	0.99
Caoutchouc (India Rubber)	0.93
Carbon disulphide	1.29
Castor-oil	0.96
Cedar, red and white, dry †	0.45
Cement, Natural (Rosendale), loose	1.04
Cement, Portland, loose	1.35
Cement, Natural, solid	2.95
Cement, Portland, solid	3.15
Chalk	2.35
Champagne	0.99
Charcoal of pines and oaks	....
Cherry, dry †	0.66
Chestnut, dry †	0.63
Chromium	5.00
Cider	1.02

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* The values given in this table for specific gravities and for weights per
AVERAGE values.

† The word "dry" in this connection indicates that the wood contains
15% of moisture. Green timbers usually weigh from one-fifth to nearly
than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

vities and Weights per Cubic Foot of Various Substances *
(Continued)

pecific gravities is pure water at 62° F., 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
....	8.12	507.0
lry.......... 1.8 to 2.1	1.90	118.5
mp, loose..........	1.01	63.0
e, 1.3 to 1.84; of Penn., 1.3 to 1.7..........	1.50	93.5
e, broken, of any size, loose, average..........	....	52 to 56
e, broken, moderately shaken..........	....	56 to 60
e, broken, heaped bushel, loose, 77 to 83 lb	....	....
e, broken, a ton loose occupies 40 to 43	....	....
us, solid, 1.2 to 1.5..........	1.35	84.0
us, solid, Cambria Co., Pa., 1.27 to 1.34.	....	79 to 84
us, broken, of any size, loose..........	....	47 to 52
us, moderately shaken..........	....	51 to 56
us, a heaped bushel, loose, 70 to 78..........	....	....
us, 1 ton occupies 43 to 48 cu ft..........	....	....
d quality..........	....	23 to 32
eaped bushel, 35 to 42 lb..........	....	....
n occupies 80 to 97 cu ft..........	....	....
.... 130 to 150	2.33	145.0
r.......... 100 to 110	1.68	105.0
red.......... 8.8 to 9.0	8.95	558.0
.... 8.9 to 9.0	8.95	558.0
wire.......... 8.8 to 9.0	8.89	554.5
....	8.72	543.6
.... 8.6 to 8.9	8.82	550.0
....	8.23	513.0
....	0.24	15.0
.... 3.92 to 4.01	3.96	247.5
.... 1.04 to 1.10	1.07	66.8
can, dry †..........	0.55	34.3
....	0.75	46.8
†..........	0.51	31.8
loam, perfectly dry, loose..........	....	72 to 80
loam, perfectly dry, shaken..........	....	82 to 92
loam, perfectly dry, rammed..........	....	90 to 100
loam, slightly moist, loose..........	....	70 to 76
loam, more moist, loose..........	....	66 to 68
loam, more moist, shaken..........	....	75 to 90
loam, more moist, packed..........	....	90 to 100
loam, as soft, flowing mud..........	....	104 to 112
loam, as soft, flowing mud, well-pressed	....	110 to 120
....	1.22	76.0
....	1.09	68.0
....	0.076	4.7
....	0.56	35.0
....	0.80	50.0
....	2.70	168.5

ven in this table for specific gravities and for weights per cubic foot are

ry" in this connection indicates that the wood contains not more than Green timbers usually weigh from one-fifth to nearly one-half more y building-timbers, *tolerably seasoned, one-sixth more.*

Specific Gravities and Weights per Cubic Foot of Various Subs
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1
Emery	4.00
Fats	0.93
Feldspar	2.57
Filbert-tree, dry †	0.60
Fir, Douglas (see Douglas Fir).	
Flint	2.63
Gamboge	1.20
Garnet 3.4 to 4.3	3.85
Glass, optical	3.45
Glass, flint	3.00
Glass, white	2.89
Glass, plate	2.80
Glass, green	2.67
Glass, floor, heavy	2.53
Glass, window	2.50
Gneiss (see Granites).	
Gold, pure	19.50
Gold, hammered, native	19.40
Gold, cast	19.258
Granites and gneiss, Connecticut, Greenwich	2.84
California, Penryn (hornblende)	2.77
New York	2.74
Maryland, Port Deposit	2.72
Massachusetts, Quincy (hornblende)	2.70
Wisconsin, Athelstane	2.70
Georgia, Lithornia and Stone Mountain	2.69
Minnesota	2.68
California, Rocklin (muscovite)	2.68
Rhode Island, Westerley	2.67
Connecticut, New London	2.66
New Hampshire, Keene	2.66
Maine, Hallowell	2.65
New Hampshire, Concord	2.65
Vermont, Barre	2.65
Wisconsin, Montello	2.64
Colorado, Georgetown (biotite)	2.63
Maine, Fox Island	2.63
Massachusetts, Rockport	2.61
Graphite	2.26
Gravel, dry	1.79
Gravel, wet	2.00
Greenstone, trap 2.8 to 3.2	3.00
Grindstone	2.14
Gum arabic	1.32
Gun-metal (see Bronze).	
Gunpowder (granular)	1.00
Gutta-percha	0.98

* The values given in this table for specific gravities and for weights per cu
AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not
15% of moisture. Green timbers usually weigh from one-fifth to nearly one
than dry; ordinary building-timbers, tolerably seasoned, one-sixth more

avities and Weights per Cubic Foot of Various Substances *
(Continued)

specific gravities is pure water at 62° F., er 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
er of Paris....................2.24 to 2.30	2.27	141.6
, unburned	2.30	143.4
ned, in lump	1.80	112.0
ler, solid	2.55	159.0
ler, loose	0.97	60.5
ler, shaken	1.03	64.0
see Larch).		
stacks, about 512 cu ft per ton	....	....
†	0.42	26.2
ut, dry †	0.89	55.6
cer-nut, dry	0.85	53.1
bark, dry	0.81	50.6
neg, dry	0.78	48.7
n, dry	0.78	48.7
rnut, dry	0.77	48.1
r, dry	0.73	45.6
	0.76	47.4
	1.45	90.5
	1.69	105.5
..............................3.0 to 3.5	3.25	202.7
..............................88 to .914	.89	56.0
	4.94	308.0
	22.12	1 379.0
..............................6.9 to 7.4	7.2	448.9
ndry, cold	7.21	450.0
ndry, molten	6.94	433.0
	7.70	480.0
	1.88	117.0
	0.57	35.6
	2.20	137.2
	2.65	165.2
matack, dry †	0.55	34.3
	0.91	58.7
cial, cast	11.36	708.0
cial, sheet	11.40	710.8
	11.42	713.0
	10.40	648.8
dry †....................0.65 to 1.33	0.99	41 to 84
nois	2.57	160.4
	2.50	155.8
	2.685	167.4
	2.44	152.1
	2.655	165.6
	2.32	144.8
	2.71	169.0
mestones	2.60	162.1
	0.935	58.3

iven in this table for specific gravities and for weights per cubic foot are

dry " in this connection indicates that the wood contains not more than
. Green timbers usually weigh from one-fifth to nearly one-half more
ry *building-timbers, tolerably seasoned, one-sixth* more.

Specific Gravities and Weights per Cubic Foot of Various Substances
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1
Locust, dry †	0.71
Magnesite	3.0
Magnesium, pure	1.72
Mahogany ....0.56 to 1.06	0.81
Manganese, pure	8.00
Manganese, ore, red	4.01
Manganese, ore, black	3.45
Marble, average ....2.6 to 164.4	2.6
domestic,	
New York	2.83
California	2.75
Georgia	2.73
Vermont, Dorset	2.66
foreign,	
Parian	2.84
African	2.80
Carrara	2.72
Biscayan	2.71
British	2.71
French	2.65
Marl	2.10
Masonry, brickwork (see Brickwork)	
Masonry, concrete, stone	2.33
Masonry, concrete, cinder	1.68
Masonry, granite, dressed	2.64
Masonry, granite, rubble in cement	2.48
Masonry, limestone, dressed	2.60
Masonry, marble, dressed for buildings	2.72
Masonry, sandstone	2.41
Mastic, gum resin	0.85
Mercury, at 32° F	13.62
Mica ....2.75 to 3.1	2.93
Milk, at 32° F	1.032
Molybdenum, pure	8.63
Mortar, lime	1.65
Mortar, cement	1.68
Mud, dry, close	....
Mud, wet, moderately pressed	....
Mud, wet, fluid	....
Mulberry-tree, dry †	0.75
Naptha-oil, wood, at 32° F	0.85
Nickel	8.56
Oak, live, dry † ....0.88 to 1.02	0.95
Oak, white, dry † ....0.66 to 0.88	0.77
Oak, red and black, dry †	....
Ochre	3.50
Olive-oil, 32° F	0.916

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* The values given in this table for specific gravities and for weights per cubic
AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not
15% of moisture. Green timbers usually weigh from one-fifth to nearly
than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

and Weights per Cubic Foot of Various Substances *
(Continued)

gravities is pure water at 62° F., Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
	2.25	140.3
	2.15	134.0
	1.34	83.6
	0.71	44.3
	11.80	735.8
	0.95	59.3
	0.88	54.9
	0.67	41.8
	0.72	45.0
	0.878	54.8
	0.63	39.3
, dry	0.61	38.1
	0.53	33.1
f, dry	0.51	31.8
y	0.50	31.2
	0.44	27.5
	0.38	23.7
	1.08	67.0
ypsum)	2.25	140.3
	21.50	1 340.6
	2.10	131.0
	0.47	29.3
	2.30	143.4
	2.76	172.3
	2.26	141.0
	0.865	54.0
	0.92	57.4
	2.65	165.3
†	0.71	44.3
	8.94	557.5
	1.09	68.0
	2.60	162.0
	0.73	45.6
	1.10	68.6
	0.93	58.0
	3.90	243.0
k bushel, Syracuse, N. Y., 56 lb	....	45.0
	2.02	122 to 130
perfectly dry and loose	....	90 to 106
voids full of water	....	118 to 129
very large and small grains, dry	....	117.0
	2.44	152.1
gmeadow	2.49	155.4
nd	2.50	156.0
2.40 to 2.70	2.60	162.1
ille	2.40	149.7
	2.63	164.2
	2.60	162.0

this table for specific gravities and for weights per cubic foot are

this connection indicates that the wood contains not more than
n timbers usually weigh from one-fifth to nearly one-half more
ding-timbers, tolerably *seasoned, one-sixth more.*

Specific Gravities and Weights per Cubic Foot of Various Substanc
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1
Sandstone, (continued)	
Ohio	2.22
Michigan	2.35
Wisconsin	2.22
Minnesota	2.25
Colorado	2.33
California, Angel Island	2.73
Shales, red or black	2.60
Silica	2.66
Silver	10.50
Slate	2.81
Snow, freshly fallen	....
Snow, moistened, compacted by rain	....
Soapstone	2.73
Sodium	0.978
Spelter	7.10
Spirit, rectified	0.824
Spruce	0.40
Steel, cast	7.9
Steel, wrought	7.85
Sugar	1.60
Sycamore, dry	0.58
Talc	2.81
Tallow	0.94
Tamarack	0.38
Tar	1.00
Teak	0.70
Tellurium	6.27
Tiles, solid	2.20
Tin, rolled	7.40
Tin, cast	7.30
Tin, molten	7.02
Trap (see Basalt).	
Tungsten	19.129
Turpentine	0.87
Type-metal, cast	10.45
Uranium	18.49
Urine	1.02
Vinegar	1.08
Walnut, black, dry	0.60
Water, pure rain, distilled, at 32° F., barometer 30 in.	....
Water, pure rain, distilled, at 62° F., barometer 30 in.	1.00
Water, pure rain, distilled, at 212° F., barometer 30 in.	....
Water, sea .... 1.026 to 1.030	1.028
Wax (see Beeswax).	
Willow	0.49
Wine	1.01
Zinc or spelter .... 6.8 to 7.2	7.00

* The values given in this table for specific gravities and for weights per cu
AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not
15% of moisture. Green timbers usually weigh from one-fifth to nearly one
than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

IRE-GAUGES AND METAL-DATA

e is a method of designating the diameter of wires or the thick-
etal by the numbers of a table arranged on a certain fixed basis.
e present time several gauges, resulting in great confusion.
: 402, gives the diameters of the gauges in common use. The
in this country is the United States standard gauge, described
t is used by most of the manufacturers of sheet iron and steel
he Brown & Sharpe gauge is commonly used for designating
ires (see page 1424); also for sheet copper and brass. The
nd Wire Company uses the old Washburn & Moen gauge for
d iron wire and also for wire nails. The sectional areas for
ven on page 1426. When placing orders for sheets and wire,
to specify the weight per square or linear foot or the thickness
ousandths of an inch. The gauge for steel wire, used by the
Sons Company, is given on page 403, and the circular-mil
87.

Weights in Pounds per Square Foot of Sheets of Wrought Iron, Steel, Copper, and Brass *

Thickness by American (Brown & Sharpe) gauge †

No. of gauge	Thickness in inches	Iron	Steel	Copper	Brass
0000	0.46	18.40	18.77	20.84	19.69
000	0.4096	16.39	16.71	18.56	17.53
00	0.3648	14.59	14.88	16.53	15.61
0	0.3249	12.99	13.25	14.72	13.90
1	0.2893	11.57	11.80	13.11	12.38
2	0.2576	10.31	10.51	11.67	11.03
3	0.2294	9.18	9.36	10.39	9.82
4	0.2043	8.17	8.34	9.26	8.74
5	0.1819	7.28	7.42	8.24	7.7
6	0.1620	6.48	6.61	7.34	6.9
7	0.1443	5.77	5.89	6.54	6.1
8	0.1285	5.14	5.24	5.82	5.5
9	0.1144	4.58	4.67	5.18	4.9
10	0.1019	4.08	4.16	4.62	4.3
11	0.0907	3.63	3.70	4.11	3.8
12	0.0808	3.23	3.30	3.66	3.
13	0.0720	2.88	2.94	3.26	3.
14	0.0641	2.56	2.61	2.90	2.
15	0.0571	2.28	2.33	2.59	2.
16	0.0508	2.03	2.07	2.30	2.
17	0.0453	1.81	1.85	2.05	1.
18	0.0403	1.61	1.64	1.83	1.
19	0.0359	1.44	1.46	1.63	1.
20	0.0320	1.28	1.30	1.45	1.
21	0.0285	1.14	1.16	1.29	1.
22	0.0253	1.01	1.03	1.15	1.
23	0.0226	0.903	0.921	1.02	0.
24	0.0201	0.804	0.820	0.911	0.
25	0.0179	0.716	0.730	0.811	0.
26	0.0159	0.638	0.650	0.722	0.
27	0.0142	0.568	0.579	0.643	0.
28	0.0126	0.506	0.516	0.573	0.
29	0.0113	0.450	0.459	0.510	0
30	0.0100	0.401	0.409	0.454	0
31	0.0089	0.357	0.364	0.404	0
32	0.0080	0.318	0.324	0.360	0
33	0.0071	0.283	0.289	0.321	0
34	0.0063	0.252	0.257	0.286	0
35	0.0056	0.224	0.229	0.254	0
Specific gravity...........		7.704	7.85	8.72	
Weight per cubic foot......		480.00	489.60	543.6	5
Weight per cubic inch.....		0.2778	0.2833	0.3146	

* This table is taken from the handbook of the Cambria Steel Company.

† As there are many gauges in use differing from each other and as even the t of a certain specified gauge are not assumed to be the same by all manufacture for sheets and wire should always state the weight in pounds per square f thickness in thousandths of an inch.

	sq ft, lb	1 ft long, lb	1 ft long, lb	sq ft, lb	1 ft long, lb	1 ft long, lb	sq ft, lb	1 ft long, lb	1 ft long, lb	
1/32	1.86	0.005	0.004	1.44	0.004	0.003	1.36	0.004	0.003	1/32
1/16	3.72	0.019	0.015	2.89	0.015	0.012	2.71	0.014	0.011	1/16
3/32	5.58	0.044	0.034	4.33	0.034	0.027	4.06	0.032	0.025	3/32
1/8	7.44	0.078	0.061	5.77	0.060	0.047	5.42	0.056	0.044	1/8
5/32	9.30	0.121	0.095	7.20	0.094	0.074	6.75	0.088	0.069	5/32
3/16	11.20	0.174	0.137	8.66	0.135	0.106	8.13	0.127	0.100	3/16
7/32	13.00	0.237	0.187	10.10	0.184	0.144	9.50	0.173	0.136	7/32
1/4	14.90	0.310	0.244	11.50	0.240	0.189	10.80	0.226	0.177	1/4
5/16	18.60	0.485	0.381	14.40	0.376	0.295	13.50	0.353	0.277	5/16
3/8	22.30	0.698	0.548	17.30	0.541	0.425	16.30	0.508	0.399	3/8
7/16	26.00	0.950	0.746	20.30	0.736	0.578	19.00	0.691	0.543	7/16
1/2	29.80	1.240	0.974	23.10	0.962	0.755	21.70	0.903	0.709	1/2
9/16	33.50	1.570	1.230	26.00	1.220	0.955	24.30	1.140	0.900	9/16
5/8	37.20	1.940	1.520	28.90	1.500	1.180	27.10	1.410	1.110	5/8
11/16	40.90	2.340	1.840	31.70	1.820	1.430	29.80	1.700	1.340	11/16
3/4	44.60	2.790	2.190	34.60	2.160	1.700	32.50	2.030	1.600	3/4
13/16	48.30	3.270	2.570	37.50	2.550	1.990	35.20	2.380	1.870	13/16
7/8	52.10	3.800	2.980	40.40	2.940	2.310	37.90	2.760	2.170	7/8
15/16	56.00	4.370	3.420	43.30	3.380	2.650	40.60	3.180	2.490	15/16
1	59.50	4.960	3.900	46.20	3.850	3.020	43.30	3.610	2.840	1
1 1/8	66.90	6.270	4.920	52.00	4.870	3.820	48.70	4.570	3.600	1 1/8
1 1/4	74.40	7.750	6.090	57.70	6.010	4.720	54.20	5.640	4.430	1 1/4
1 3/8	81.80	9.370	7.370	63.50	7.280	5.720	59.60	6.820	5.370	1 3/8
1 1/2	89.30	11.200	8.770	69.30	8.650	6.800	65.00	8.120	6.380	1 1/2
1 5/8	96.70	13.100	10.30	75.10	10.200	7.980	70.40	9.530	7.490	1 5/8
1 3/4	104.00	15.200	11.90	80.80	11.800	9.250	75.90	11.100	8.680	1 3/4
1 7/8	112.00	17.500	13.70	86.60	13.500	10.600	81.30	12.700	9.970	1 7/8
2	119.00	19.800	15.60	92.30	15.400	12.100	86.70	14.400	11.300	2

Sizes and Weights of Smooth Steel Wire*

As made by the American Steel and Wire Company

No. of gauge†	Diameters			Sectional area, sq in	Weight		No. of feet per pound
	Fractions of inch	Decimals of inch	Millimeters		Pounds per 100 feet	Pounds per mile	
000000	....	0.4615	11.72	0.16728	56.81	2999.0	1.76
......	7/16	0.4375	11.11	0.15033	51.05	2696.0	[illegible]
00000	....	0.4305	10.93	0.14556	49.43	2610.0	2.02
......	13/32	0.40625	10.32	0.12962	44.02	2324.0	2.27
0000	....	0.3938	10.00	0.12180	41.36	2184.0	2.41
......	3/8	0.3750	9.525	0.11045	37.51	1980.0	2.66
000	....	0.3625	9.2075	0.10321	35.05	1851.0	2.85
......	11/32	0.34375	8.731	0.092806	31.52	1664.0	3.17
00	....	0.3310	8.407	0.086049	29.22	1543.0	3.42
......	5/16	0.3125	7.938	0.076699	26.05	1375.0	3.84
0	....	0.3065	7.785	0.073782	25.06	1323.0	3.99
1	....	0.2830	7.188	0.062902	21.36	1128.0	4.68
......	9/32	0.28125	7.144	0.062126	21.10	1114.0	4.74
2	....	0.2625	6.668	0.054119	18.38	970.4	5.44
......	1/4	0.2500	6.350	0.049087	16.67	880.2	5.99
3	....	0.2437	6.190	0.046645	15.84	836.4	6.31
4	....	0.2253	5.723	0.039867	13.54	714.8	7.38
......	7/32	0.21875	5.556	0.037583	12.76	673.9	7.83
5	....	0.2070	5.258	0.033654	11.43	603.4	8.750
6	....	0.1920	4.877	0.028953	9.832	519.2	10.17
......	3/16	0.1875	4.763	0.027612	9.377	495.1	[illegible]
7	....	0.1770	4.496	0.024606	8.356	441.2	11.97
8	....	0.1620	4.115	0.020612	7.000	369.6	14.28
......	5/32	0.15625	3.969	0.019175	6.512	343.8	[illegible]
9	....	0.1483	3.767	0.017273	5.866	309.7	17.05
10	....	0.1350	3.429	0.014314	4.861	256.7	20.57
......	1/8	0.125	3.175	0.012272	4.168	220.0	24.00
11	....	0.1205	3.061	0.011404	3.873	204.5	25.82
12	....	0.1055	2.680	0.0087417	2.969	156.7	33.69
......	3/32	0.09375	2.381	0.0069029	2.344	123.8	42.65
13	....	0.0915	2.324	0.0065755	2.233	117.9	44.78
14	....	0.0800	2.032	0.0050266	1.707	90.13	[illegible]
15	....	0.0720	1.829	0.0040715	1.383	73.01	[illegible]
16	1/16	0.0625	1.588	0.0030680	1.042	55.01	[illegible]
17	....	0.0540	1.372	0.0022902	0.7778	41.07	128.60
18	....	0.0475	1.207	0.0017721	0.6018	31.77	[illegible]
19	....	0.0410	1.041	0.0013203	0.4484	23.67	[illegible]
20	....	0.0348	0.8839	0.00095115	0.3230	17.05	309.60
21	....	0.0317	0.8052	0.00078924	0.2680	14.15	373.10
......	1/32	0.03125	0.7938	0.00076699	0.2605	13.75	383.90
22	....	0.0286	0.7264	0.00064242	0.2182	11.52	458.40
23	....	0.0258	0.6553	0.00052279	0.1775	9.37	563.50
24	....	0.0230	0.5842	0.00041548	0.1411	7.45	708.70

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* For iron wire, the values in columns 6 and 7 should be multiplied by 0.98 and
copper wire, by 1.12.

For other wire-gauges see pages 402, 403, 1387 and 1424.

† American Steel and Wire Company's gauge.

Wire Manufactured by the American Steel and Wire Company

, Nos. 0000 to 18.
one-wire or weaving-wire, Nos. 16 to 47.
ket-wire, Nos. 0 to 18.
e-wire, Nos. 16 to 40.
wire, finished with great care as regards roundness and exactness
18 to 50.
wire, Nos. 00000 to 18.
ire, ½-in diameter, down to No. 26.
edle-steel wire, Nos. 12 to 25.
ARKET-WIRE applies to the ordinary and most used forms of Bes-
ED, BRIGHT, GALVANIZED, TINNED and COPPERED wires.

-Iron-Wire Strand. The diameter, list-price per 100 ft, weight
d approximate breaking-load in pounds for this wire is given in
apter XI.

Weights and Areas of Square and Round Bars and Circumferen Round Steel Bars*

Weights are for steel, at 489.6 lb per cu ft

Thickness or diameter, in	Weight of □ bar 1 ft long, lb	Weight of ○ bar 1 ft long, lb	Area of □ bar, sq in	Area of ○ bar, sq in
1/16	0.013	0.010	0.0039	0.0031
5/64	0.021	0.016	0.0061	0.0048
3/32	0.030	0.023	0.0088	0.0069
7/64	0.041	0.032	0.0120	0.0094
1/8	0.053	0.042	0.0156	0.0123
9/64	0.067	0.053	0.0198	0.0155
5/32	0.083	0.065	0.0244	0.0192
11/64	0.100	0.079	0.0295	0.0232
3/16	0.120	0.094	0.0352	0.0276
13/64	0.140	0.110	0.0413	0.0324
7/32	0.163	0.128	0.0479	0.0376
15/64	0.187	0.147	0.0549	0.0431
1/4	0.213	0.167	0.0625	0.0491
17/64	0.240	0.188	0.0706	0.0554
9/32	0.269	0.211	0.0791	0.0621
19/64	0.300	0.235	0.0881	0.0692
5/16	0.332	0.261	0.0977	0.0767
11/32	0.402	0.316	0.1182	0.0928
3/8	0.478	0.376	0.1406	0.1104
13/32	0.561	0.441	0.1650	0.1296
7/16	0.651	0.511	0.1914	0.1503
15/32	0.747	0.587	0.2197	0.1726
1/2	0.850	0.668	0.2500	0.1963
17/32	0.960	0.754	0.2822	0.2217
9/16	1.076	0.845	0.3164	0.2485
19/32	1.199	0.941	0.3525	0.2769
5/8	1.328	1.043	0.3906	0.3068
11/16	1.607	1.262	0.4727	0.3712
3/4	1.913	1.502	0.5625	0.4418
13/16	2.245	1.763	0.6602	0.5185
7/8	2.603	2.044	0.7656	0.6013
15/16	2.989	2.347	0.8789	0.6903

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* Adapted from the 1912 Edition of the Handbook of the Cambria Ste Johnstown, Pa.

eights and Areas of Square and Round Steel Bars *

Weights are for steel, at 489.6 lb per cu ft

□	○			□		○	
Weight per foot, lb	Area, sq in	Weight per foot, lb	Thickness, in	Area, sq in	Weight per foot, lb	Area, sq in	Weight per foot, lb
3.400	0.785	2.670	3	9.000	30.60	7.069	24.03
3.838	0.887	3.014	1/16	9.379	31.89	7.366	25.04
3.303	0.994	3.379	1/8	9.766	33.20	7.670	26.08
4.795	1.108	3.766	3/16	10.16	34.55	7.980	27.13
5.312	1.227	4.173	1/4	10.56	35.92	8.296	28.20
5.857	1.353	4.600	5/16	10.97	37.31	8.618	29.30
6.428	1.485	5.049	3/8	11.39	38.73	8.946	30.42
7.026	1.623	5.518	7/16	11.82	40.18	9.281	31.56
7.650	1.767	6.008	1/2	12.25	41.65	9.621	32.71
8.301	1.918	6.520	9/16	12.69	43.14	9.968	33.90
8.978	2.074	7.051	5/8	13.14	44.68	10.32	35.09
9.682	2.237	7.604	11/16	13.60	46.24	10.68	36.31
10.41	2.405	8.178	3/4	14.06	47.82	11.05	37.56
11.17	2.580	8.773	13/16	14.54	49.42	11.42	38.81
11.95	2.761	9.388	7/8	15.02	51.05	11.79	40.10
12.76	2.948	10.02	15/16	15.50	52.71	12.18	41.40
13.60	3.142	10.68	4	16.00	54.40	12.57	42.73
14.46	3.341	11.36	1/16	16.50	56.11	12.96	44.07
15.35	3.547	12.06	1/8	17.02	57.85	13.36	45.44
16.27	3.758	12.78	3/16	17.54	59.62	13.77	46.83
17.22	3.976	13.52	1/4	18.06	61.41	14.19	48.24
18.19	4.200	14.28	5/16	18.60	63.23	14.61	49.66
19.18	4.430	15.07	3/8	19.14	65.08	15.03	51.11
20.20	4.666	15.86	7/16	19.69	66.95	15.47	52.58
21.25	4.909	16.69	1/2	20.25	68.85	15.90	54.07
22.33	5.157	17.53	9/16	20.82	70.78	16.35	55.59
23.43	5.412	18.40	5/8	21.39	72.73	16.80	57.12
24.56	5.673	19.29	11/16	21.97	74.70	17.26	58.67
25.71	5.940	20.20	3/4	22.56	76.71	17.72	60.25
26.90	6.213	21.12	13/16	23.16	78.74	18.19	61.84
28.10	6.492	22.07	7/8	23.77	80.81	18.67	63.46
29.34	6.777	23.04	15/16	24.38	82.89	19.15	65.10

rom the 1912 Edition of the Handbook of the Cambria Steel Company,

Weights and Areas of Square and Round Steel Bars * (Co

Weights are for steel, at 489.6 lb per cu ft

Thickness, in	□		○		Thickness, in	□	
	Area, sq in	Weight per foot, lb	Area, sq in	Weight per foot, lb		Area, sq in	Weight per foot, lb
5	25.00	85.00	19.64	66.76	7	49.00	166.6
1/16	25.63	87.14	20.13	68.44	1/4	52.56	178.7
1/8	26.27	89.30	20.63	70.14	1/2	56.25	191.3
3/16	26.91	91.49	21.14	71.86	3/4	60.06	204.2
1/4	27.56	93.72	21.65	73.60	8	64.00	217.6
5/16	28.22	95.96	22.17	75.37	1/4	68.06	231.4
3/8	28.89	98.23	22.69	77.15	1/2	72.25	245.6
7/16	29.57	100.5	23.22	78.95	3/4	76.56	260.3
1/2	30.25	102.8	23.76	80.77	9	81.00	275.4
9/16	30.94	105.2	24.30	82.62	1/4	85.56	290.9
5/8	31.64	107.6	24.85	84.49	1/2	90.25	306.8
11/16	32.35	110.0	25.41	86.38	3/4	95.06	323.2
3/4	33.06	112.4	25.97	88.29	10	100.0	340.0
13/16	33.79	114.9	26.54	90.22	1/4	105.1	357.2
7/8	34.52	117.4	27.11	92.17	1/2	110.3	374.9
15/16	35.25	119.9	27.69	94.14	3/4	115.6	392.9
6	36.00	122.4	28.27	96.14	11	121.0	411.4
1/8	37.52	127.6	29.47	100.2	1/4	126.6	430.3
1/4	39.06	132.8	30.68	104.3	1/2	132.3	449.6
3/8	40.64	138.2	31.92	108.5	3/4	138.1	469.4
1/2	42.25	143.6	33.18	112.8	12	144.0	489.6
5/8	43.89	149.2	34.47	117.2	.....	.....	......
3/4	45.56	154.9	35.79	121.7	.....	.....	......
7/8	47.27	160.8	37.12	126.2	.....	.....	......

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* Adapted from the 1912 Edition of the Handbook of the Cambria Johnstown, Pa.

Weights in Pounds of Flat Rolled Steel Bars

PER LINEAR FOOT

One cubic foot of steel weighs 489.6 lb

For thicknesses from 1/16 in to 9/16 in and widths from ¼ in to ¾ in

ess, es	Width of bar, inches								
	¼	5/16	3/8	7/16	½	9/16	5/8	11/16	¾
	0.053	0.066	0.080	0.093	0.106	0.120	0.133	0.146	0.159
	0.066	0.083	0.100	0.116	0.133	0.149	0.166	0.183	0.199
	0.080	0.100	0.120	0.139	0.159	0.179	0.199	0.219	0.239
	0.093	0.116	0.139	0.163	0.186	0.209	0.232	0.256	0.279
	0.106	0.133	0.159	0.186	0.212	0.239	0.266	0.292	0.319
	0.120	0.149	0.179	0.209	0.239	0.269	0.299	0.329	0.359
	0.133	0.166	0.199	0.232	0.266	0.299	0.332	0.365	0.398
	0.146	0.183	0.219	0.256	0.292	0.329	0.365	0.402	0.438
	0.159	0.199	0.239	0.279	0.319	0.359	0.398	0.438	0.478
	0.173	0.216	0.259	0.302	0.345	0.388	0.432	0.475	0.518
	0.186	0.232	0.279	0.325	0.372	0.418	0.465	0.511	0.558
	0.199	0.249	0.299	0.349	0.398	0.448	0.498	0.548	0.598
	0.213	0.266	0.319	0.372	0.425	0.478	0.531	0.584	0.638
	0.226	0.282	0.339	0.395	0.452	0.508	0.564	0.621	0.677
	0.239	0.299	0.359	0.418	0.478	0.538	0.598	0.657	0.717
	0.252	0.315	0.379	0.442	0.505	0.568	0.631	0.694	0.757
	0.266	0.332	0.398	0.465	0.531	0.598	0.664	0.730	0.797
	0.279	0.349	0.418	0.488	0.558	0.628	0.697	0.767	0.827
	0.292	0.365	0.438	0.511	0.584	0.657	0.730	0.804	0.877
	0.305	0.382	0.458	0.535	0.611	0.687	0.764	0.840	0.916
	0.319	0.398	0.478	0.558	0.638	0.717	0.797	0.877	0.956
	0.332	0.415	0.498	0.581	0.664	0.747	0.830	0.913	0.996
	0.345	0.432	0.518	0.604	0.691	0.777	0.863	0.950	1.04
	0.359	0.448	0.538	0.628	0.717	0.807	0.896	0.986	1.08
	0.372	0.465	0.558	0.651	0.744	0.837	0.930	1.02	1.12
	0.385	0.481	0.578	0.674	0.770	0.867	0.963	1.06	1.16
	0.398	0.498	0.598	0.697	0.797	0.896	0.996	1.10	1.20
	0.412	0.515	0.618	0.721	0.823	0.926	1.03	1.13	1.24
	0.425	0.531	0.638	0.744	0.850	0.956	1.06	1.17	1.28
	0.438	0.548	0.657	0.767	0.877	0.986	1.10	1.21	1.31
	0.452	0.564	0.677	0.790	0.903	1.02	1.13	1.24	1.35
	0.465	0.581	0.697	0.813	0.930	1.05	1.16	1.28	1.39
	0.478	0.598	0.717	0.837	0.956	1.08	1.20	1.31	1.43

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 1 to 5 in

Thickness, inches	Width of bar, inches						
	1	1¼	1½	1¾	2	2¼	2½
1/16	0.21	0.26	0.32	0.37	0.43	0.48	0.53
⅛	0.42	0.53	0.64	0.75	0.85	0.96	1.06
3/16	0.63	0.79	0.96	1.11	1.28	1.44	1.59
¼	0.85	1.06	1.28	1.49	1.70	1.91	2.12
5/16	1.06	1.33	1.59	1.86	2.12	2.39	2.65
⅜	1.28	1.59	1.92	2.23	2.55	2.87	3.19
7/16	1.49	1.86	2.23	2.60	2.98	3.35	3.72
½	1.70	2.12	2.55	2.98	3.40	3.83	4.25
9/16	1.92	2.39	2.87	3.35	3.83	4.30	4.78
⅝	2.12	2.65	3.19	3.72	4.25	4.78	5.31
11/16	2.34	2.92	3.51	4.09	4.67	5.26	5.84
¾	2.55	3.19	3.83	4.47	5.10	5.75	6.38
13/16	2.76	3.45	4.14	4.84	5.53	6.21	6.90
⅞	2.98	3.72	4.47	5.20	5.95	6.69	7.44
15/16	3.19	3.99	4.78	5.58	6.38	7.18	7.97
1	3.40	4.25	5.10	5.95	6.80	7.65	8.50
1 1/16	3.61	4.52	5.42	6.32	7.22	8.13	9.03
1⅛	3.83	4.78	5.74	6.70	7.65	8.61	9.57
1 3/16	4.04	5.05	6.06	7.07	8.08	9.09	10.10
1¼	4.25	5.31	6.38	7.44	8.50	9.57	10.63
1 5/16	4.46	5.58	6.69	7.81	8.93	10.04	11.16
1⅜	4.67	5.84	7.02	8.18	9.35	10.52	11.69
1 7/16	4.89	6.11	7.34	8.56	9.78	11.00	12.22
1½	5.10	6.38	7.65	8.93	10.20	11.48	12.75
1 9/16	5.32	6.64	7.97	9.30	10.63	11.95	13.28
1⅝	5.52	6.90	8.29	9.67	11.05	12.43	13.81
1 11/16	5.74	7.17	8.61	10.04	11.47	12.91	14.34
1¾	5.95	7.44	8.93	10.42	11.90	13.40	14.88
1 13/16	6.16	7.70	9.24	10.79	12.33	13.86	15.40
1⅞	6.38	7.97	9.57	11.15	12.75	14.34	15.94
1 15/16	6.59	8.24	9.88	11.53	13.18	14.83	16.47
2	6.80	8.50	10.20	11.90	13.60	15.30	17.00

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 3½ to 7½ in

Width of bar, inches								
3½	4	4½	5	5½	6	6½	7	7½
0.75	0.85	0.96	1.06	1.17	1.28	1.39	1.49	1.60
1.49	1.70	1.92	2.13	2.34	2.55	2.77	2.98	3.19
2.23	2.55	2.87	3.19	3.51	3.83	4.14	4.46	4.78
2.98	3.40	3.83	4.25	4.67	5.10	5.53	5.95	6.36
3.72	4.25	4.78	5.31	5.84	6.38	6.90	7.44	7.97
4.47	5.10	5.74	6.38	7.02	7.65	8.29	8.93	9.57
5.20	5.95	6.70	7.44	8.18	8.93	9.67	10.41	11.16
5.95	6.80	7.65	8.50	9.35	10.20	11.05	11.90	12.75
6.70	7.65	8.61	9.57	10.52	11.48	12.43	13.39	14.34
7.44	8.50	9.57	10.63	11.69	12.75	13.81	14.87	15.94
8.18	9.35	10.52	11.69	12.85	14.03	15.20	16.36	17.53
8.93	10.20	11.48	12.75	14.03	15.30	16.58	17.85	19.13
9.67	11.05	12.43	13.81	15.19	16.58	17.95	19.34	20.72
10.41	11.90	13.39	14.87	16.36	17.85	19.34	20.83	22.32
11.16	12.75	14.34	15.94	17.53	19.13	20.72	22.32	23.91
11.90	13.60	15.30	17.00	18.70	20.40	22.10	23.80	25.50
12.65	14.45	16.26	18.06	19.87	21.68	23.48	25.29	27.10
13.39	15.30	17.22	19.13	21.04	22.95	24.87	26.78	28.68
14.13	16.15	18.17	20.19	22.21	24.23	26.24	28.26	30.28
14.87	17.00	19.13	21.25	23.38	25.50	27.62	29.75	31.88
15.62	17.85	20.08	22.32	24.54	26.78	29.01	31.23	33.48
16.36	18.70	21.04	23.38	25.71	28.05	30.39	32.72	35.06
17.10	19.85	21.99	24.44	26.88	29.33	31.77	34.21	36.66
17.85	20.40	22.95	25.50	28.05	30.60	33.15	35.70	38.26
18.60	21.25	23.91	26.57	29.22	31.88	34.53	37.19	39.84
19.34	22.10	24.87	27.63	30.39	33.15	35.91	38.67	41.44
20.08	22.95	25.82	28.69	31.55	34.43	37.30	40.16	43.03
20.83	23.80	26.78	29.75	32.73	35.70	38.68	41.65	44.63
21.57	24.65	27.73	30.81	33.89	36.98	40.05	43.14	46.22
22.31	25.50	28.69	31.87	35.06	38.25	41.44	44.63	47.82
23.06	26.35	29.64	32.94	36.23	39.53	42.82	46.12	49.41
23.80	27.20	30.60	34.00	37.40	40.80	44.20	47.60	51.00

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 8 to 12 in

Thickness, inches	Width of bar, inches							
	8	8½	9	9½	10	10½	11	11
1/16	1.70	1.81	1.91	2.02	2.13	2.23	2.34	2.
1/8	3.40	3.61	3.82	4.04	4.25	4.46	4.68	4.
3/16	5.10	5.42	5.74	6.06	6.38	6.70	7.02	7.
1/4	6.80	7.22	7.65	8.08	8.50	8.92	9.34	9.
5/16	8.50	9.03	9.56	10.10	10.62	11.16	11.68	12.
3/8	10.20	10.84	11.48	12.12	12.75	13.39	14.03	14.
7/16	11.90	12.64	13.40	14.14	14.88	15.62	16.36	17.
1/2	13.60	14.44	15.30	16.16	17.00	17.85	18.70	19.
9/16	15.30	16.26	17.22	18.18	19.14	20.08	21.02	22.
5/8	17.00	18.06	19.13	20.19	21.25	22.32	23.38	24.
11/16	18.70	19.86	21.04	22.21	23.38	24.54	25.70	26.
3/4	20.40	21.68	22.96	24.23	25.50	26.78	28.05	29.
13/16	22.10	23.48	24.86	26.24	27.62	29.00	30.40	31.
7/8	23.80	25.30	26.78	28.26	29.75	31.24	32.72	34.
15/16	25.50	27.10	28.69	30.28	31.88	33.48	35.06	36.
1	27.20	28.90	30.60	32.30	34.00	35.70	37.40	39.
1 1/16	28.90	30.70	32.52	34.32	36.12	37.92	39.71	41.
1 1/8	30.60	32.52	34.43	36.34	38.25	40.17	42.08	44.
1 3/16	32.30	34.32	36.34	38.36	40.38	42.40	44.42	46
1 1/4	34.00	36.12	38.26	40.37	42.50	44.63	46.76	48
1 5/16	35.70	37.93	40.16	42.40	44.64	46.86	49.08	51.
1 3/8	37.40	39.74	42.08	44.41	46.75	49.08	51.42	53
1 7/16	39.10	41.54	44.00	46.44	48.88	51.32	53.76	56
1 1/2	40.80	43.35	45.90	48.45	51.00	53.55	56.10	58
1 9/16	42.50	45.16	47.82	50.48	53.14	55.78	58.42	61
1 5/8	44.20	46.96	49.73	52.49	55.25	58.02	60.78	63
1 11/16	45.90	48.76	51.64	54.51	57.38	60.24	63.10	65
1 3/4	47.60	50.58	53.56	56.53	59.50	62.48	65.45	68
1 13/16	49.30	52.38	55.46	58.54	61.62	64.70	67.80	70
1 7/8	51.00	54.20	57.38	60.56	63.75	66.94	70.12	73
1 15/16	52.70	56.00	59.29	62.58	65.88	69.18	72.46	75
2	54.40	57.80	61.20	64.60	68.00	71.40	74.80	78

for Estimating the Weight of any Piece of Wrought Iron, Steel or Cast Iron

Iron.

bic foot of wrought iron weighs........................ 480 lb
uare foot, one inch thick, weighs...................... 40 lb
uare inch, one foot long, weighs....................... 3⅓ lb

he weight per square foot of sheet iron, multiply the thickness in
.

he weight per linear foot of bars of any section, multiply the cross-
ea in square inches by 3⅓.

bic foot of steel weighs............................... 489.6 lb
just 2% more than wrought iron.)
uare foot, one inch thick, weighs...................... 40.8 lb
uare inch, one foot long, weighs....................... 3.4 lb

he weight per linear foot, of bars of any section, multiply the cross-
a in square inches by 3.4; or, if the weight is known, the exact sec-
nay be obtained by dividing by 3.4.

.

bic foot of cast iron weighs........................... 450 lb
uare foot, one inch thick, weighs...................... 37½ lb
uare inch, one foot long, weighs....................... 3⅛ lb
bic inch weighs.. 0.26 lb

t of irregular castings must be estimated by the cubic inch.

Rules for Weights of Castings

the weight of the pattern by 18 for cast iron, 13 for brass, 19 for lead,
11.4 for zinc; the product is the weight of the casting.

Reduction for Round Cores and Core-Prints

ultiply the square of the diameter by the length of the core in inches,
luct multiplied by 0.017 is the weight of the pine core to be deducted
ight of the pattern.

Shrinkage in Castings

makers' Rule
Cast iron..⅛
Brass.....3/16
Lead.....⅛
Tin......1/12
Zinc......5/16
of an inch longer per linear foot

Weights of Square Cast-Iron Columns in Pounds per Lin

a□b $2a + 2b$ †	Thickness of metal, inches						
	5/8 in., lb	3/4 in., lb	7/8 in., lb	1 in., lb	1 1/8 in., lb	1 1/4 in., lb	1 1/2 in., lb
12	18.6	21.1	23.3	25.0	26.4	27.3	28.1
14	22.5	25.8	28.7	31.3	33.4	35.1	37.5
16	26.4	30.5	34.2	37.5	40.4	43.0	46.9
18	30.3	35.2	39.7	43.8	47.4	50.8	56.3
20	34.2	39.8	45.1	50.0	54.5	58.6	65.6
22	38.1	44.5	50.6	56.3	61.5	66.4	75.0
24	42.0	49.2	56.1	62.5	68.5	74.2	84.4
26	45.9	53.9	61.5	68.8	75.6	82.0	93.8
28	49.8	58.6	67.0	75.0	82.6	89.8	103.1
30	53.7	63.3	72.5	81.3	89.6	97.7	112.5
32	57.6	68.0	77.9	87.5	96.7	105.5	121.9
34	61.5	72.7	83.4	93.8	103.7	113.3	131.3
36	65.4	77.3	88.9	100.0	110.7	121.1	140.6
38	69.3	82.0	94.3	106.3	117.8	128.9	150.0
40	73.2	86.7	99.8	112.5	124.8	136.7	159.4
42	77.1	91.4	105.3	118.8	131.8	144.5	168.8
44	81.0	96.1	110.8	125.0	138.8	152.3	178.1
46	84.9	100.8	116.2	131.3	145.9	160.2	187.5
48	88.8	105.5	121.7	137.5	152.9	168.0	196.9
50	92.8	110.2	127.2	143.8	159.9	175.8	206.3
52	96.7	114.8	132.6	150.0	167.0	183.6	215.6
54	100.6	119.5	138.1	156.3	174.0	191.4	225.0
56	104.5	124.2	143.6	162.5	181.0	199.2	234.4
58	108.4	128.9	149.0	168.8	188.1	207.0	243.8
60	112.3	133.6	154.5	175.0	195.1	214.9	253.2
62	116.2	138.3	160.0	181.3	202.1	222.7	262.5
64	120.1	143.0	165.4	187.5	209.2	230.5	271.9
66	124.0	147.7	170.9	193.8	216.2	238.3	281.3
68	127.9	152.3	176.4	200.0	223.2	246.1	290.6
70	131.8	157.0	181.8	206.3	230.3	253.9	300.0
72	135.7	161.7	187.3	212.5	237.3	261.7	309.4
74	139.6	166.4	192.8	218.8	244.3	269.5	318.8
76	143.5	171.1	198.3	225.0	251.3	277.3	328.1
78	147.4	175.8	203.7	231.3	258.4	285.2	337.5
80	151.3	180.5	207.2	237.5	265.4	293.0	346.9

* Birkmire.

† a and b = either side, outside measurement. $2a + 2b$ = number.
been made in this table for corners counted twice.

Example. What is the weight per linear foot of a 12 by 16
column?

Solution. $2a + 2b = 24 + 32 = 56$. Opposite this number, un
metal, we find 162.5, or weight per linear foot of a column 12 by 16

Note. For flanges, brackets, etc., calculate the cubical content
multiply by 0.26; cast iron averages 450 lb per cu ft.

Weights per Linear Foot of Circular Cast-Iron Columns *†

le er, s	Thickness of metal, inches							
	½ in, lb	⅝ in, lb	¾ in, lb	⅞ in, lb	1 in, lb	1⅛ in, lb	1¼ in, lb	1⅜ in, lb
	12.3	14.6	16.60	18.30	19.6	......	......	......
	17.2	21.0	24.00	27.00	29.5	32.1	33.8	35.4
	22.1	27.0	31.30	35.50	39.3	43.0	46.0	49.0
	27.0	33.0	39.00	44.00	49.1	54.1	58.3	62.4
	32.0	39.1	46.00	53.00	59.0	65.1	70.6	76.1
	36.8	45.3	53.40	61.20	69.1	76.1	83.1	89.5
	41.7	51.4	61.10	70.00	78.6	87.1	95.1	103.1
	46.6	57.5	68.13	78.41	88.4	98.0	107.4	116.4
	51.6	64.0	75.50	87.10	98.2	109.1	120.1	130.1
	56.5	70.0	82.87	96.10	108.0	120.0	132.1	143.5
	61.4	76.0	90.23	104.20	118.1	131.2	144.2	157.1
	66.3	82.1	97.60	113.20	128.1	142.0	156.5	170.4
	71.2	88.2	104.96	121.40	137.5	153.3	169.4	184.1
	76.1	94.4	112.33	130.10	147.3	164.3	181.0	197.4
	81.0	100.5	120.10	139.10	157.1	175.4	193.3	211.0
	86.0	107.0	127.00	147.00	167.0	186.4	206.0	224.4
	91.0	113.0	134.40	156.00	177.1	197.5	218.1	238.0
	96.0	119.0	142.10	164.30	186.6	208.8	230.1	251.5
	100.6	125.0	149.10	173.10	196.6	219.6	242.4	265.0
	105.6	131.2	156.50	181.50	206.2	230.6	255.0	278.0
	110.5	137.3	164.10	190.10	216.1	242.0	267.0	292.0
	115.4	143.5	171.20	199.00	226.0	253.0	279.2	305.4

le er, s	Thickness of metal, inches							
	1½ in, lb	1⅝ in, lb	1¾ in, lb	1⅞ in, lb	2 in, lb	2⅛ in, lb	2¼ in, lb	2⅜ in, lb
	......	......	......	......	......	......	......	......
	......	......	......	......	......	......	......	......
	51.54	54.1	55.84	57.5	......	......	......	......
	66.30	69.9	73.02	76.0	78.6	80.84	82.83	......
	81.00	85.6	90.20	94.3	98.2	101.70	105.00	107.84
	95.80	101.8	107.40	112.8	117.8	122.60	127.00	131.20
	110.50	117.7	124.60	131.2	137.5	143.40	149.10	154.50
	125.20	133.7	142.00	149.6	157.1	164.30	171.20	177.80
	140.00	149.6	159.00	168.0	176.8	185.20	193.30	201.10
	154.70	165.6	176.00	186.4	196.4	206.00	215.40	224.40
	169.40	181.5	193.30	204.8	216.0	226.90	237.50	247.70
	184.10	197.4	210.50	223.2	235.7	247.70	259.60	271.10
	198.90	213.4	227.70	241.6	255.3	268.20	281.70	294.40
	213.50	229.4	244.90	260.0	274.9	289.50	303.70	317.70
	228.30	245.3	262.00	278.4	294.5	310.30	325.80	341.00
	243.00	261.3	279.20	296.8	314.2	331.20	348.00	364.30
	257.70	277.2	296.40	315.2	338.8	352.10	370.00	387.70
	272.50	293.2	313.60	333.6	353.4	372.90	392.10	411.00
	287.20	309.0	330.80	352.1	373.1	393.80	414.20	434.30
	302.00	325.1	348.00	370.5	393.0	414.60	436.30	457.60
	316.70	341.0	365.10	388.9	412.3	435.50	458.40	481.00
	331.40	357.0	382.30	407.3	432.0	456.40	480.50	504.20

nire.

table is arranged for the weight of plain shaft. For brackets, flanges, etc., cal-
cubical contents and multiply *by 0.26.*

Weight of Cast-Iron Plates

Weights, in Pounds, of Cast-Iron Plates One Inch T

Calculated at 450 lb per cu ft

Length, inches	Width, inches							
	6 in, lb	8 in, lb	10 in, lb	12 in, lb	14 in, lb	16 in, lb	18 in, lb	20 in lb
4	6.25	8.3	10.4	12.5	14.6	16.6	18.7	20.
6	9.37	12.5	15.6	18.7	21.8	25.0	28.1	31.
8	12.50	16.6	20.8	25.0	29.1	33.3	37.4	41.
10	15.60	20.8	26.0	31.2	36.4	41.6	46.8	52.
12	18.70	25.0	31.2	37.5	43.7	49.9	56.2	62.
14	21.80	29.2	36.4	43.7	51.0	58.2	65.5	72.
16	24.90	33.3	41.6	50.0	58.2	66.6	74.9	83.
18	28.10	37.5	46.8	56.2	65.5	74.9	84.2	93.
20	31.20	41.6	52.0	62.3	72.8	83.2	93.6	104.
22	34.30	45.8	57.2	68.6	80.1	91.5	103.0	114.
24	37.50	50.0	62.4	75.0	87.4	99.8	112.3	124.
26	40.60	54.0	67.6	81.2	94.6	108.2	121.7	135.
28	43.60	58.2	72.8	87.5	101.9	116.5	131.0	145.
30	46.80	62.4	78.0	93.7	109.2	124.8	140.4	156.
32	49.80	66.6	83.2	100.0	116.5	133.1	150.3	166.
36	56.10	75.0	93.6	112.5	131.0	150.0	168.4	187.

For larger plates take size of plate ONE-HALF smaller and multi
a plate 28 by 32 in will weigh twice as much as one 14 by 32 in.
or less than one inch in thickness multiply weight of plate by thi

Approximate Weights of Square-Ribbed Cast-Iron Colum

The following table, giving the weight of cast-iron column-base
when estimating the steel and iron in tall buildings.*

Size of square base, in	Weight, lb	Size of square base, in	Weight, lb
22×22	600	32×32	1 340
24×24	750	34×34	1 450
26×26	880	36×36	1 600
28×28	1 020	38×38	1 720
30×30	1 180	40×40	1 850

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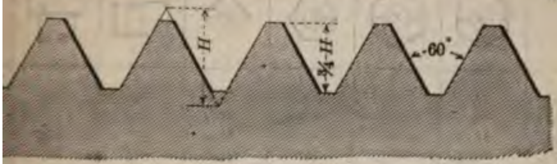
* H. G. Tyrrell, in Architects and Builders Magazine, January

Screw-Threads, Nuts, and Bolt-Heads

Standard Screw-Threads

nended by Franklin Institute, December 15, 1864, and adopted by Navy De-
of the United States; by the R. R. Master Mechanics' and Master Car-Builders'
ns; by Jones & Laughlin Steel Company; and by many other of the promi-
neering and mechanical establishments of the country.

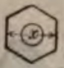
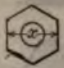
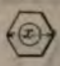
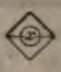
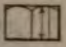


Angle of thread 60°. Flat at top and bottom ⅛ of pitch.

Threads per inch	Diam at root of thread, in	Area at root of thread, sq in	Diam of screw, in	Threads per inch	Diam at root of thread, in	Area at root of thread, sq in
20	0.185	0.027	2	4½	1.712	2.302
18	0.240	0.045	2¼	4½	1.962	3.023
16	0.294	0.068	2½	4	2.176	3.719
14	0.344	0.093	2¾	4	2.426	4.620
13	0.400	0.126	3	3½	2.629	5.428
12	0.454	0.162	3¼	3½	2.879	6.510
11	0.507	0.202	3½	3¼	3.100	7.548
10	0.620	0.302	3¾	3	3.317	8.641
9	0.731	0.420	4	3	3.567	9.963
8	0.837	0.550	4¼	2⅞	3.798	11.329
7	0.940	0.694	4½	2¾	4.028	12.753
7	1.065	0.893	4¾	2⅝	4.256	14.226
6	1.160	1.057	5	2½	4.480	15.763
6	1.284	1.295	5¼	2½	4.730	17.572
5½	1.389	1.515	5½	2⅜	4.953	19.267
5	1.491	1.746	5¾	2⅜	5.203	21.262
5	1.616	2.051	6	2¼	5.423	23.098

d **Bolt-Heads** are determined by the following rules, which apply to both
hexagon nuts:
ameter of rough nut = 1½ × diam of bolt + ⅛ in.
ameter of finished nut = 1½ × diam of bolt + 1/16 in.
s of rough nut = diam of bolt.
s of finished nut = diam of bolt − 1/16 in.
ameter of rough head = 1½ × diam of bolt + ⅛ in.
ameter of finished head = 1½ × diam of bolt + 1/16 in.
s of rough head = ½ short diam of head.
s of finished head = diam of bolt − 1/16 in.
diameter of a hexagon nut may be determined by multiplying the short di-
1.155, and the long diameter of a square nut by multiplying the short diameter

Standard Dimensions of Nuts and Bolt-Heads

Diam of bolt	Short diam, rough 	Short diam, finished 	Long diam, rough 	Long diam, rough 	Thickness, rough. Nut 
1/4	1/2	7/16	37/64	7/10	1/4
5/16	19/32	17/32	11/16	19/12	5/16
3/8	11/16	5/8	51/64	63/64	3/8
7/16	25/32	23/32	9/10	1 7/64	7/16
1/2	7/8	13/16	1	1 15/64	1/2
9/16	31/32	29/32	1 1/8	1 23/64	9/16
5/8	1 1/16	1	1 7/32	1 1/2	5/8
3/4	1 1/4	1 3/16	1 7/16	1 49/64	3/4
7/8	1 7/16	1 3/8	1 21/32	2 1/32	7/8
1	1 5/8	1 9/16	1 7/8	2 19/64	1
1 1/8	1 13/16	1 3/4	2 3/32	2 9/16	1 1/8
1 1/4	2	1 15/16	2 5/16	2 53/64	1 1/4
1 3/8	2 3/16	2 1/8	2 17/32	3 3/32	1 3/8
1 1/2	2 3/8	2 5/16	2 3/4	3 23/64	1 1/2
1 5/8	2 9/16	2 1/2	2 31/32	3 5/8	1 5/8
1 3/4	2 3/4	2 11/16	3 3/16	3 57/64	1 3/4
1 7/8	2 15/16	2 7/8	3 13/32	4 5/32	1 7/8
2	3 1/8	3 1/16	3 5/8	4 27/64	2
2 1/4	3 1/2	3 7/16	4 1/16	4 61/64	2 1/4
2 1/2	3 7/8	3 13/16	4 1/2	5 31/64	2 1/2
2 3/4	4 1/4	4 3/16	4 29/32	6	2 3/4
3	4 5/8	4 9/16	5 3/8	6 17/32	3
3 1/4	5	4 15/16	5 13/16	7 1/16	3 1/4
3 1/2	5 3/8	5 5/16	6 7/64	7 39/64	3 1/2
3 3/4	5 3/4	5 11/16	6 21/32	8 1/8	3 3/4
4	6 1/8	6 1/16	7 3/32	8 41/64	4
4 1/4	6 1/2	7 7/16	7 9/16	9 3/16	4 1/4
4 1/2	6 7/8	6 13/16	7 31/32	9 3/4	4 1/2
4 3/4	7 1/4	7 3/16	8 13/32	10 1/4	4 3/4
5	7 5/8	7 9/16	8 27/32	10 49/64	5
5 1/4	8	7 15/16	9 9/32	11 23/64	5 1/4
5 1/2	8 3/8	8 5/16	9 23/32	11 7/8	5 1/2
5 3/4	8 3/4	8 11/16	10 5/32	12 3/8	5 3/4
6	9 1/8	9 1/16	10 19/32	12 15/16	6

Weights of One Hundred Bolts With Square Heads and Nuts

INCLUDES WEIGHT OF NUT

Hoopes & Townsend's List

Length under head to point in	Diameter of bolts								
	¼ in,	5/16 in,	⅜ in,	7/16 in,	½ in, lb	⅝ in, lb	¾ in, lb	⅞ in, lb	1 in, lb
	lb	lb	lb	lb					
1½	4.00	7.00	10.50	15.20	22.50	39.50	63.00	.....	.....
1¾	4.40	7.50	11.25	16.30	23.82	41.62	66.00	.....	.....
2	4.75	8.00	12.00	17.40	25.15	43.75	69.00	109.00	163
2¼	5.15	8.50	12.75	18.50	26.47	45.88	72.00	113.25	169
2½	5.50	9.00	13.50	19.60	27.80	48.00	75.00	117.50	174
2¾	5.75	9.50	14.25	20.70	29.12	50.12	78.00	121.75	180
3	6.25	10.00	15.00	21.80	30.45	52.25	81.00	126.00	185
3½	7.00	11.00	16.50	24.00	33.10	56.50	87.00	134.25	196
4	7.75	12.00	18.00	26.20	35.75	60.75	93.10	142.50	207
4½	8.50	13.00	19.50	28.40	38.40	65.00	99.05	151.00	218
5	9.25	14.00	21.00	30.60	41.05	69.25	105.20	159.55	229
5½	10.00	15.00	22.50	32.80	43.70	73.50	111.25	168.00	240
6	10.75	16.00	24.00	35.00	46.35	77.75	117.30	176.60	251
6½	.....	.....	25.50	37.20	49.00	82.00	123.35	185.00	262
7	.....	.....	27.00	39.40	51.65	86.25	129.40	193.65	273
7½	.....	.....	28.50	41.60	54.30	90.50	135.00	202.00	284
8	.....	.....	30.00	43.80	59.60	94.75	141.50	210.70	295
9	.....	.....	.....	46.00	64.90	103.25	153.60	227.75	317
10	.....	.....	.....	48.20	70.20	111.75	165.70	224.80	339
11	.....	.....	.....	50.40	75.50	120.25	177.80	261.85	360
12	.....	.....	.....	52.60	80.80	128.75	189.90	278.90	382
13	.....	.....	.....	.....	86.10	137.25	202.00	295.95	404
14	.....	.....	.....	.....	91.40	145.75	214.10	313.00	426
15	.....	.....	.....	.....	96.70	154.25	226.20	330.05	448
16	.....	.....	.....	.....	102.00	162.75	238.30	347.10	470
17	.....	.....	.....	.....	107.30	171.00	250.40	364.15	492
18	.....	.....	.....	.....	112.60	179.50	262.60	381.20	514
19	.....	.....	.....	.....	117.90	188.00	274.70	398.25	536
20	.....	.....	.....	.....	123.20	206.50	286.80	415.30	558
Per inch additional	1.37	2.13	3.07	4.18	5.45	8.52	12.27	16.70	21.82

Weights of Nuts and Bolt-Heads, in Pounds

For calculating the weight of longer bolts

Diameter of bolt, in inches		¼	⅜	½	⅝	¾	⅞
Weight of hexagon nut and head...	....	0.017	0.057	0.128	0.267	0.43	0.73
Weight of square nut and head....	....	0.021	0.069	0.164	0.320	0.55	0.88

Diameter of bolt, in inches	1	1¼	1½	1¾	2	2½	3
Weight of hexagon nut and head...	1.10	2.14	3.78	5.6	8.75	17	28.8
Weight of square nut and head....	1.31	2.56	4.42	7.0	10.50	21	36.4

Weights of Rivets and Round-Headed Bolts Without Nuts. St

POUNDS PER HUNDRED

Length, in	⅜ in diam	½ in diam	⅝ in diam	¾ in diam	⅞ in diam	1 in diam	1⅛ in diam
1¼	5.5	12.8	22.0	29.3	43.9	66.6	93.3
1½	6.3	14.2	24.1	32.4	48.2	72.1	100
1¾	7.0	15.5	26.3	35.5	52.5	77.7	107.
2	7.9	16.9	28.5	38.7	56.7	83.3	114
2¼	8.7	18.3	30.7	41.8	61.0	88.8	121
2½	9.4	19.7	32.8	44.9	65.2	94.4	128
2¾	10.2	21.1	35.0	48.0	69.5	100.	136
3	11.0	22.5	37.2	51.1	73.7	105.	143
3¼	11.7	23.9	39.3	54.3	78.0	111	150
3½	12.6	25.3	41.5	57.4	82.3	116	157
3¾	13.4	26.7	43.7	60.5	86.5	122	164
4	14.1	28.1	45.9	63.6	90.8	128	170
4¼	14.9	29.4	48.0	66.7	95.0	134	177
4½	15.7	30.8	50.2	69.9	99.3	139	185
4¾	16.5	32.2	52.4	73.0	104	145	192
5	17.2	33.6	54.5	76.1	108	150	199
5¼	18.1	35.0	56.7	79.2	112	156	206
5½	18.8	36.4	58.9	82.3	116	161	213
5¾	19.6	37.8	61.1	85.5	120	166	220
6	20.4	39.2	63.2	88.6	124	172	227
6½	21.9	42.0	67.6	95.1	133	184	241
7	23.5	44.7	71.9	101	142	195	255
7½	25.1	47.5	76.1	108	150	206	269
8	26.6	50.3	80.6	114	159	217	284
8½	28.2	53.1	85.0	120	167	227	298
9	29.8	55.9	89.3	126	176	239	312
9½	31.3	58.7	93.7	133	185	250	325
10	32.8	61.4	98.0	139	193	261	340
10½	34.5	64.2	103	145	202	272	354
11	36.0	67.0	107	151	210	284	368
11½	37.6	69.8	111	158	218	295	382
12	39.2	72.5	115	164	227	306	396
Heads......	1.8	5.8	11.1	13.6	22.6	39.0	58.

For length of shaft required to form rivet-head, see Table IV, page 420.

NAILS AND SCREWS*

Based upon the process of manufacture there are three kinds of nails on use, namely, plate or cut nails, wire nails, and clinch-nails. These ly described in the following subdivisions of this article and other data on the subject is included.

t Nails. Cut nails are made from a strip of rolled iron or steel of the ckness as the finished nail and a little wider than its length, the fiber on being parallel with the length of the nail. Special machinery cuts out in alternate wedge-shaped slices, the heads are then stamped and the finished nails dropped into the casks. Cut nails made from iron rally preferred for use in exposed positions. Cut nails are made in a f shapes to suit special uses. For ordinary use in building, nails of three shapes are made, and the nails are called COMMON NAILS, FINISH-NAILS NG-NAILS. The common nails are used for rough work, finish-nails for work, and casing-nails for flooring, matched ceiling and sometimes for ngs, although the heads are rather too large for finish-work. Cut nails ning to return to favor as they have holding power and lasting qualities to wire nails.

ads. Brads are thin nails with a small head, used for small finish, panel- , etc. They vary from ¼ to 2 in in length.

ut-Nails. Clout-nails are made with broad, flat heads, and are sold arying from ⅜ to 2½ in in length. They are used chiefly for fastening nd metal-work. Special nails are also made for lathing, slating, shing-

re Nails. These have of late years become as common as the cut nails, old at about the same price. They are said to be stronger for driving cut nails, not so liable to bend or break, especially when driven into hard nd less liable to split the wood; for these reasons they are generally by carpenters. Wire nails are made from wire, of the same section- as the shank of the nail, by a machine which cuts the wire in even heads and points them, and, when desired, also barbs them. In general classification is used for cut nails. It should be noticed that the gauge re and the shape of the head vary in the different kinds, and that some ed, others plain. The various types of wire nails are drawn ROUND, or BARBED, for the domestic trade; for export they are drawn OVAL, or DIAMOND-SHAPED, according to the country to which they are to be and its requirements. It is customary to charge 15 cents more per 100 ndard nails, BARBED, than for the same nails, SMOOTH.

nch-Nails. These are made from open-hearth or Bessemer-steel wire. nary wire nail will clinch, especially when made with DUCK-BILL or points for clinching purposes, or even otherwise, if annealed. These used only in places where it is desired to turn over the ends of the nails clinch, as in the case of battens or cleats.

ngth and Weight of Nails. The length of nails is designated by PEN-). This classification originally represented the price in English pence ails, as *2d* per 100, etc. In that sense it is of course now obsolete, but retained and is practically uniform with the various manufacturers, cut and wire nails. The weights expressed in pennies run from two sixty pennies, the larger sizes being designated by fractions of an inch.

nsed from article by Thomas Nolan in chapter on Builders' Hardware in re- on of Building Construction and Superintendence, Part II, Carpenters' Work Kidder.

in thickness, tenpenny nails should be used. Clapboardin
with sixpenny finish-nails or casing-nails. Fourpenny nai
shingling and slating, and threepenny for lathing. For sla
should be used, and they are also better for shingling. Wh
should be used may generally be left to the builder; but in
any danger of the nails being drawn out either by the war
from the pull of the nail, cut nails should be used, as they
power than the wire nails under certain conditions. It is
that a wire nail will hold more firmly when barbed than
page 1445.)

(8) **Copper and Brass Nails.** Nails are also made of c
and these are sometimes used in connection with boat-l
work, etc. One wing of the Physical Laboratory Buildin
is put together entirely with brass and copper. As the roo
use in delicate electrical work, no iron was used in their co

(9) **Cement-coated Wire Nails.** The coating consist
gums mixed by a secret formula, and put on the nails by a
involves the use of quite complicated machinery. Altho
for coated nails is among the users of packages to be shipp
market for them among builders, for construction-purpose
the coating is that it gives the nail an adhesive resistance
that of ordinary wire nails. This quality appeals especially
and users of packages to be shipped, for which strength is pa
is desirable for construction-purposes also, but the lack of h
wire nails is not so apparent in building. About 90% of th
factories and large shippers.* Cement-coated nails are qu
in laying both ordinary and parquetry-flooring. The use
special head which leaves a small hole, gives a firm floor an
Though the makers do not claim that the nails are absolute
claim that nails thus treated will resist the effects of mois
better than the uncoated wire nails. But it is when in use
quality is most evident. There is more coating on the

differing only in the heads, and are either COOLERS or SINKERS. have large flat heads; the latter, heads slightly reinforced by counter-hey are made to replace common nails, in sizes from ¼ in to 1 in, d for framing, boarding, shingling and staging, and for boxes and sults of tests made with cement-coated nails to determine their istance in comparison with the common smooth-wire nails are given

ing table shows the result of tests made at the United States Arsenal, Mass., in 1902, the wood being pine:

ative Adhesive Resistance of Common Smooth-Wire Nails and Cement-Coated Nails

were driven into the same piece and were perpendicular to the grain

Size and name	Diameter, in	Length driven,* in	Adhesive resistance,† lb
common, smooth	0.145	2½	167
coated	0.117	2½	418
, common, smooth	0.132	2¼	182
, coated	0.114	2¼	327
y, common, smooth	0.132	2	189
y, coated	0.112	2	316
ommon, smooth	0.097	1⅝	106
oated	0.092	1⅝	226

of the nails were left with their heads projecting from ¼ to ½ in.

erage of three trials.

Power of Nails. A committee appointed by the Wheeling nail-ers, a number of years ago, to test the comparative holding power of nails, published the following data, although the kind of wood is not

Pounds Required to Pull Nails Out

	Cut	Wire		Cut	Wire
nny	1 593	703	Sixpenny	383	200
	908	315	Fourpenny	286	123
y	597	227			

ing power of nails varies with the kind of wood into which they are stin T. Byrne gives the relative holding power of woods as ABOUT as hite pine, 1; yellow pine, 1.5; white oak, 3; chestnut, 1.6; beech, 3.2; ; elm, 2; basswood, 1.2.

omparative Holding Power of Cut and Wire Nails

rough tests of the comparative holding power of wire nails and cut AL LENGTHS AND WEIGHTS were made at the U. S. Arsenal in 1892 and n forty series, comprising forty sizes of nails driven in spruce wood, it hat the cut nails showed an average superiority of 60.50%, the com-howing an average superiority of 47.51% and the finishing-nails an 2.22%. In eighteen series, comprising six sizes of BOX-NAILS driven ood, in three ways the cut nails showed an average superiority of n no series of tests did *the wire nails hold as much as* the cut nails.

Quantity of Nails Required for Different Kinds of

For 1 000 shingles allow 5 lb fourpenny nails or 3½ lb threepenny
1 000 laths, 7 lb threepenny fine, or for 100 sq yd of lathing, 10 l
1 000 sq ft of beveled siding, 18 lb sixpenny
1 000 sq ft of sheathing, 20 lb eightpenny or 25 lb tenpenny
1 000 sq ft of flooring, 30 lb eightpenny or 40 lb tenpenny
1 000 sq ft of studding, 15 lb tenpenny and 5 lb twentypenny
1 000 sq ft of 1 by 2½-in furring, 12-in centers, 9 lb eightpenny
1 000 sq ft of 1 by 2½-in furring, 16-in centers, 7 lb eightpenny

Cut Steel Nails and Spikes

Sizes, lengths, and approximate number per pound
Taken from the Handbook of the Cambria Steel Com

Sizes	Length, inches	Common	Clinch	Finishing	Casing and box	Fe
2*d*	1	740	400	1 100	....	..
3*d*	1¼	460	260	880	....	..
4*d*	1½	280	180	530	420	..
5*d*	1¾	210	125	350	300	
6*d*	2	160	100	300	210	
7*d*	2¼	120	80	210	180	
8*d*	2½	88	68	168	130	
9*d*	2¾	73	52	130	107	
10*d*	3	60	48	104	88	
12*d*	3¼	46	40	96	70	
16*d*	3½	33	34	86	52	
20*d*	4	23	24	76	38	
25*d*	4¼	20	....	....	....	..
30*d*	4½	16½	....	....	30	..
40*d*	5	12	....	....	26	..
50*d*	5½	10	....	....	20	..
60*d*	6	8	....	....	16	..
......	6½	....	....	....	....	..
......	7	....	....	....	....	..

Sizes	Length, inches	Barrel	Light barrel	Slating	Sizes	Length, inches
....	⅝	750	....	....	....	¾
....	¾	600	....	....	....	⅞
....	⅞	500	....	....	2*d*	1
2*d*	1	450	....	340	3*d*	1⅛
....	1⅛	310	400	....	4*d*	1¼
3*d*	1¼	280	304	280		
....	1⅜	210	....	....	Tobacco	
4*d*	1½	190	224	220		
5*d*	1¾	....	....	180	130	
6*d*	2	....	....	....	97	
7*d*	2¼	....	....	....	85	
8*d*	2½	....	....	....	68	
9*d*	2¾	....	....	....	58	
10*d*	3	....	....	....	48	
12*d*	3¼	....	....	....	....	
16*d*	3½	....	....	....	....	

Steel-Wire Nails, Spikes, and Tacks

ZE, LENGTH, GAUGE AND APPROXIMATE NUMBER TO THE POUND

Compiled from Catalogue of American Steel and Wire Company, 1910
American Steel and Wire Company's gauge. (See page 1426.)

ommon nails and brads *			Casing-nails †		Finishing-nails†	
Length, in	Gauge	Number to pound	Gauge	Number to pound	Gauge	Number to pound
1	15	876	15½	1 010	16½	1 351
1¼	14	568	14½	635	15½	807
1½	12½	316	14	473	15	584
1¾	12½	271	14	406	15	500
2	11½	181	12½	236	13	309
2¼	11½	161	12½	210	13	238
2½	10¼	106	11½	145	12½	189
2¾	10¼	96	11½	132	12½	172
3	9	69	10½	94	11½	121
3¼	9	63	10½	87	11½	113
3½	8	49	10	71	11	90
4	6	31	9	52	10	62
4½	5	24	9	46	....	....
5	4	18	8	35	....	....
5½	3	14				
6	2	11				

Spikes ‡		
Length, in	Gauge	Number to pound
3	6	41
3¼	6	38
3½	5	30
4	4	23
4½	3	17
5	2	13
5½	1	10
6	1	9
7	5/16"	7
8	3/8"	4
9	3/8"	3½
10	3/8"	3
12	3/8"	2½

Shingle-nails			
Size	Length, in	Gauge	Number to pound
3*d*	1¼	13	429
3½*d*	1⅜	12½	345
4*d*	1½	12	274
5*d*	1¾	12	235
6*d*	2	12	204
7*d*	2¼	11	139
8*d*	2½	11	125
9*d*	2¾	11	114
10*d*	3	10	83
Fine nails			
2*d*	1	16½	1 351
3*d*	1⅛	15	778
4*d*	1½	14	473
2*d* extra fine	1	17	1 560
3*d* extra fine	1⅛	16	1 015

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non brads differ from common nails only in the head and point.
ths are the same as common nails for corresponding size.
s are made with chisel-points and diamond points; also with convex heads and

Steel-Wire Nails (Continued)

Clinch-nails				Fence-nails *		Slati
Size	Length, in	Gauge	Number to pound	Gauge	Number to pound	Gaug
2*d*	1	14	710			12
3*d*	1¼	13	429	No 5 smallest size		10½
4*d*	1½	12	274			10½
5*d*	1¾	12	235	10	142	10
6*d*	2	11	157	10	124	9
7*d*	2¼	11	139	9	92	Barbed
8*d*	2½	10	99	9	82	
9*d*	2¾	10	90	8	62	¾"×N
10*d*	3	9	69	7	50	⅞"×N
12*d*	3¼	9	62	6	40	1"×N
16*d*	3½	8	49	5	30	1⅛"×N
20*d*	4	7	37	4	23	1¼"×N

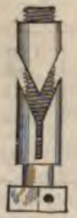
* Length same as clinch-nails of corresponding size.

† Roofing-nails are designated by the length, not by PENNY. These
lengths up to 2 in.

Wire Tacks

Title, ounce	Length, in	Number per pound	Title, ounce	Length, in	Number per pound	Title, ounce	L
1	⅛	16 000	4	7/16	4 000	14	
1½	3/16	10 666	6	9/16	2 666	16	
2	¼	8 000	8	⅝	2 000	18	
2½	5/16	6 400	10	11/16	1 600	20	
3	⅜	5 333	12	¾	1 333	22	
......	......	......	......	......	......	24	

Wire carpet-tacks are made polished, blued, tinned, or coppered; th
sterers' and bill-posters' or railroad tacks.

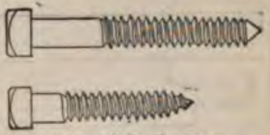


Expansion-bolt

Expansion-Bolts. These are commonly
wood or iron to masonry that is already b
drilled in the masonry of such size that the ex
fit closely, and when the bolt is screwed up
and binds firmly in the masonry. The illustr
Evans expansion-bolt, which is also furnished
bolts. There are other forms of expansion-bolt
From experiments on expansion-bolts it was
holding capacity was 264 lb per sq in when e
Portland-cement mortar, 843 per sq in whe
sulphur and 485 lb per sq in when embedde
average working unit-stresses it is safe to use a
the values given. When the work is exposed to rain or moistur

ıs the acid which results will rust the metal and will also tend to
he masonwork at the point of entrance of the bolt.

The substitution of screws for nails in building operations is a
re of modern work. Trimming hardware of all descriptions is put
rs, and a great deal of panel-work, inside finish, etc., is put together
Stop-beads, the casings of plumbing-fixtures, etc., should be fas-
crews, as well as all kinds of store and office-fixtures, and cabinet-
al, except where the joints are glued. Screws are also largely used
niture. They present a neater appearance than nails, have greater
r and are less apt to injure the material if it should be removed and
y making holes for the screws with a bit, all danger of splitting the
ed. The ordinary type of screw has a gimlet-point by which it can
the wood without the aid of a bit. The heads are made in various
different uses. Screws are made ordinarily of steel, but sometimes
ronze. The latter sort are used for screwing in place finished hard-
ame material, and have heads finished to correspond with the trim-
l screws, also, are finished with blue, bronze, lacquered, galvanized,
rface, to match the cheaper class of trimmings. The galvanized
in building operations at the seashore. Screws with blue surface,
SCREWS, are generally used with japanned hardware and for stop-
herever a cheap round-headed screw is desired. Silver, nickel, and
crews are also manufactured for use in connection with similar
Steel screws for wood are made in twenty different lengths, varying
in, and each length of screw has from six to eighteen varieties in
ere being in all thirty-one dif-
s; so that altogether there are
et about two hundred and fifty
es of ordinary screws used for
The most common shapes are
flat head, round head and oval
oval-head screw is tapered for
ng but is slightly rounded on top.
ond-point steel screws are made
r driving with a hammer. These can be driven with a hammer
length into any hard wood, and then held by one or two turns as
the ordinary screw. In ordering screws both the length and
he gauge or diameter of the shank, the material and finish, and
hich they are to be put, should be given.



Lag and Coach-screws

Metal have the same diameter throughout and the threads are V-

Screws. The sizes of screws are given in length in inches and the
e gauge, the gauge denoting the diameter. Thus, a 1-in No. 12 screw
nd 0.2158 in in diameter. The gauge-numbers range from 0 to 30
ths from ¼ to 6 in. The lengths vary by eighths of an inch up to
ters of an inch up to 3 in and by halves of an inch up to 5 in. Screws
½ in long are made in about sixteen different gauge-numbers. Table
02, gives the diameter to four places in decimals of an inch of the
rew-gauge. It should be noticed that, unlike the ordinary wire-
0 of the screw-gauge indicates the diameter of the smallest screw
ımeter of the screw increases with the number of the gauge.

ws and Coach-Screws are large, heavy screws used where great
required, as in heavy framing, and *for fixing ironwork* to timber.

Lag-screws with conical point are made with diameters of 5/16, 3/8, 7/
3/4, and 1 in, and in lengths from 1½ to 12 in; coach-screws in diamet
to 3/4 in and in lengths from 1½ to 12 in. For putting in lag-sc
should be bored which has a diameter a little greater than the unthr
of the screw and it should be bored to a depth corresponding to the l
unthreaded shank. A second hole should then be bored at the bo
first hole of a diameter somewhat less than that of the threaded sha
depth of about half its length.

Holding Power of Lag-Screws

Tests made by A. J. Cox, University of Iowa, 1891, quoted by Kent,

Kind of wood	Size of screw, in	Size of hole bored, in	Length in wood, in	Maximu resist-ance, lb
Seasoned white oak	5/8	1/2	4½	8 037
Seasoned white oak	9/16	7/16	3	6 480
Seasoned white oak	1/2	3/8	4½	8 780
Yellow-pine stick	5/8	1/2	4	3 800
White cedar, unseasoned	5/8	1/2	4	3 405

Hoopes & Townsend give the force required to draw screws out c
as follows:

Screw	½ in	5/8 in	3/4 in	7
Wood, depth	3½ in	4 in	4 in	5
Force, pounds	4 960	6 000	7 685	11

Wooden-screws are sold by the gross, lag-screws and coach-s
pound.

DATA ON EXCAVATING

Excavating is almost invariably measured by the cubic yard of
measuring excavations of irregular depth see page 65. For comp
tents of wells and cesspools, the circular area in square feet may be
the table on page 51, and this circular area multiplied by the dep
give the contents in cubic feet. The cost of excavating and rem
ordinarily made up of the following items:

(1) Loosening the earth for the shovelers;
(2) Loading by shovels into carts or barrows;
(3) Hauling or wheeling it away, including emptying and retur
(4) Spreading it out on the dump;

For every large job, such as railroad-work, it is also necessary t
lowance for keeping the hauling-road in repair, for sharpening and
and for carts, harness, superintendence and water-carriers. W
excavated can be spread over the ground immediately surroundin
tion the loosened dirt may be removed by scrapers without shov

Data for Estimating Cost of Loosening Earth. Two men with
team of horses will loosen from 20 to 30 cu yd of strong, heavy s

yd of ordinary loam. One man with a pick will loosen 1⅓ yd clay or cemented gravel, 4 yd of common loam, or 6 yd of light

uantity of LOOSENED EARTH that a man can shovel into a cart

nd	2.0 cu yd
avy soils	1.7 cu yd
	1.0 cu yd

when loosened swells to from 1⅕ to 1½ times its original bulk

of vehicles used for moving excavated materials is about as

ws	3 to 4 cu ft
lump-carts	18 to 22 cu ft
dump-wagons	27 to 45 cu ft*
rs	3 to 7 cu ft
ers	10 to 17 cu ft
on rails	27 to 80 cu ft

al Length of Haul with drag-scrapers is about 150 ft; with , 500 ft; with wheelbarrows, 250 ft; with one-horse dump-carts, verage speed of horses is given as about 200 ft per minute.

e data for estimating ‡ the cost of excavating may be found in er's Handbooks.

rth, Sand and Gravel. For general calculations the following nay be taken:

ft of chalk weigh 1 ton	19 cu ft of gravel weigh 1 ton
ft of clay weigh 1 ton	22 cu ft of sand weigh 1 ton
ft of earth weigh 1 ton	

on. A cubic yard of rock, in place, when broken up by blasting wheelbarrows or carts, will occupy a space of about 1⅘ cu yd; e cost of hauling or removal is about 50% more than for dirt.

r at $1 per day, the actual cost for loosening hard rock, including owder, etc., will average about 45 cents per cubic yard, in place, ry circumstances. In practice it will generally range between 30 pending on the position of the strata, hardness, toughness, water lerations. Soft shales and other allied rocks may frequently be and plough as low as 15 to 20 cents, while on the other hand shal- ery tough rock with an unfavorable position of strata, especially f excavations, may cost $1 per cu yd, or even considerably more. f average hard rock requires about ¼ to ⅓ lb of powder per cu yd, nature of the rock, the position of the strata, etc., may increase ore. Soft rock frequently requires more powder than hard. A er will drill 8 to 10 ft in depth of holes about 2½ ft deep and 2 in y in average hard rock, at from 12 to 18 cents per ft." §

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load for two-horse wagons such as are commonly used for hauling dirt, s from 1¼ to 1½ cu yd.

ocket-Book, by A. T. Byrne.

ndbook of Cost Data, by H. P. Gillette.

gineer's *Pocket-Book, J. C. Trautwine.*

chisel or any other tool except a hammer for dressin
work costs considerably more than hammer-dressed

Measurement of Stonework. Rough stone fr
under two classifications: rubble-stone and dimens
the pieces of irregular size most easily obtained from
cutting into ashlar 12 in or less in height and about
be of a certain size, to SQUARE over 24 in each way a
ness, is called DIMENSION-STONE. The price of th
four times the price of RUBBLE. Rubble is gener
load. Footings and flagging are usually sold by t
stone by the cubic foot. In Boston, granite blocks
sold by the ton.

In Estimating on the Cost of Stonework put into a
with different localities, and even among contractor
sion-stone footings, that is, squared stones 2 ft or
measured by the square foot. If built of large ru
footings are measured in with the wall, allowance be
of the footings. Rubblework is almost universally
16½ cu ft. The author has been unable to find
perch of 24¾ cu ft is used by stone-masons. In Phil
sections of Illinois, 22 cu ft are called a perch. Ra
ured by the cubic yard. When stonework is let b
cubic feet to the perch should be stated in the contr
also, whether or not openings are to be deducted.
made for openings of less than 70 superficial feet.

Data for Estimating Cost.† The price of com
the quarry will vary from 55 cts to $1.65 per ton, fr
delivery, according to the cost of quarrying, transp
is probably a fair average.

A ton of most of the different kinds of stones wi
perches.

The cost of laying one perch of stone may be estim t from $2.75 to $4.50 per perch, $3.50 a perch being a fair average; nortar, from $3.50 to $4.50 per perch.

ashlar depends very largely upon the kind of stone used and the to be brought. The price of the rough stock on the cars at the ry may vary from 75 cts to $1.35 per cu ft for granite and from for sandstones and limestones, depending largely upon cost of 1 cu ft of stone should make 2 sq ft of ashlar, at least. Some t stone especially suitable for ashlar and sell it at about 30 cts per s 12 in high.

utting ashlar, with stone-cutters' wages at $4 per day, will average er sq ft for soft stones, from 15 to 20 cts per sq ft for hard sandstones, and from 25 to 30 cts for granite. The cost of setting ashom 10 cts per sq ft to 25 cts for soft stones or 30 cts for granite, average price for sandstones and limestones.

cut-stone trimmings depends so largely upon the kind of stone impossible to give prices that would be of very much service. igures, however, may serve as a general guide in forming a rough rices if anything being probably a little above the cost of the nost localities.

for Sidewalks, ordinary stock, natural surface, 3 in thick, with o line, in lengths, along walk, from 3 to 5 ft, will cost, for a 3-ft cts per sq ft, or if 2 in thick, 7 cts; for a 4-ft walk, 10 cts; and for cts per sq ft. The cost of laying all sizes will average about 4 cts above figures do not include the cost of hauling.

by 24-in granite will cost at the quarry from 30 to 35 cts per lin ft; ting will cost from 12 to 14 cts additional; and the cost of freight ist also be added.

one. The following figures show the approximate cost of cut arious uses:

, size 8 by 10 ft, edges and top bush-hammered, per square asure	$0.75
, size 5 by 5 ft, select stock, edges clean-cut, natural top, ot	0.45
y 12 in, clean-cut, per linear foot	1.35
5 by 12 in, clean-cut, per linear foot	0.80
by 8 in, clean-cut, per linear foot	0.45
5 by 8 in, clean-cut, per linear foot	0.60
o in, clean-cut, per linear foot	0.65
2 in, clean-cut, per linear foot	1.25
by 12 in, clean-cut, per linear foot	1.25
1 in, clean-cut, per linear foot	1.20
1 in, rock-face edges and top, per linear foot	0.50
5 in, rock-face edges and top, per linear foot	0.35
8 in, rock-face edges and top, per linear foot	0.40
tock, 7 by 14 in, per linear foot	1.10
thick, per square foot	0.50

of cut stone above given must be added the cost of setting, which, s, steps, etc., will be about 10 cts per linear foot, and for windowt 5 cts per linear foot. For fitting, about 10 cts per cu ft, and for oints after the pieces are set in place, about 5 cts per cu ft should

DATA ON BRICKS AND BRICKWORK*

Clay Bricks. The word brick as commonly used refers to a block
clay, molded into the required shape and burned in a kiln; and, un
cently, practically all bricks were made from clay. At the present
ever, bricks are also made from sand and lime. Clay bricks may
classified as common bricks, face-bricks, fire-bricks and paving-bri
the process of manufacture, bricks are classified as soft-mud brick
bricks, dry-pressed bricks and repressed bricks.

Soft-Mud Bricks are made by tempering clay with water until it b
and plastic and then pressing it into molds either by hand or by
Practically all handmade bricks are soft-mud bricks. Soft-mud bric
REPRESSED to make face-bricks.

Stiff-Mud Bricks are machine-made. The clay is first ground
enough water is added to make a stiff mud. The stiff clay is forced th
or dies in the machine in a continuous stream, which is cut up automa
pieces the size either of the end or side of the brick. If the opening i
the end of the brick, the bricks are END-CUT BRICKS; if of the size of th
brick, they are SIDE-CUT BRICKS. Stiff-mud bricks can readily be di
from soft-mud bricks by their appearance. As good if not better br
made by the soft-mud process as by the stiff-mud process, and in
States the soft-mud bricks are probably the stronger. As far as
observation has extended in the Western States, the stiff-mud bricks
preferable to those made by the soft-mud process. Stiff-mud bricks
heavier than soft-mud bricks or hand-made bricks.

Dry-pressed Bricks are made almost entirely for face-work, altho
localities dry-pressed bricks are also used as common bricks. Hydra
bricks are dry-pressed.

Molded Bricks are always dry-pressed. Very fine bricks are m
process.

Burning of Bricks. Bricks made by any of the above processes
burned in a kiln. According to their position in the kiln, commo
designated ARCH-BRICKS or hard-burned bricks, RED BRICKS or
bricks, and SALMON BRICKS or soft bricks. As a rule, salmon bricks
use in an exterior or bearing-wall.

Color of Bricks. The color of bricks depends principally upon
of iron, lime, or magnesia in the clay. A large proportion of oxide
a clear bright red. Magnesia produces a brown color, and when in
of iron, a light-drab color. Dry-pressed bricks are often colored
either by mixing clays of different composition, or by mixing minera
the finely ground clay.

Fire-Bricks are ordinarily made from a mixture of flint clay and
They are usually white, or white mixed with brown, in color and are
lining of furnaces, fireplaces and tall chimneys.

Paving-Bricks are very hard bricks, usually vitrified or annealed
much more expensive than common bricks and are seldom used in b

Size and Weight of Clay Bricks. In this country there is no legal
the SIZE OF BRICKS, and the dimensions vary with the maker and a

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* For a complete description of clay bricks, their process of manufacture,
of all kinds of brickwork, see Chapter VII, Part I, of Building Constructio
intendence, by F. E. Kidder.

ommon standard sizes are 8¼ by 4 by 2½ in and 8¼ by 4 by 2¼ in.
England States the common brick averages about 7¾ by 3¾ by 2¼ in.
he Western States common bricks measure about 8½ by 4⅛ by 2½ in,
knesses of the walls measure about 9, 13, 18 and 22 in for thicknesses
and 2½ bricks. The sizes of all common bricks vary considerably in
ording to the degree to which they are burned; the hard bricks being
$\frac{3}{16}$ in smaller than the salmon bricks. In England the common
8¾ by 4⅜ by 2¾ in. Pressed bricks or face-bricks are more uniform
ost of the manufacturers use the same size of mold. The prevailing
ssed bricks are 8⅜ by 4⅛ by 2⅜ and 8⅜ by 4 by 2¼ in. Pressed
so made 1½ in thick and 12 by 4 by 1½ in, those of the latter size be-
y termed ROMAN BRICKS OR TILES.

HT OF BRICKS varies considerably with the quality of the clay from
are made, and also, of course, with their size. Common bricks
ut 4½ lb each, and pressed bricks vary from 5 to 5½ lb each. For
TH OF BRICKS and brickwork, see Chapter V. The FIRE-BRICKS are
ious forms to suit the required work. A straight brick measures
2½ in and weighs about 7 lb. To secure the best results fire-bricks
id in the same clay from which they are manufactured, this being
water into a thin paste. The thinner the joint, the better the wall
eat. For PAVING-BRICKS the size and weight vary according to the
to the requirements of the specifications. The STANDARD bricks are
8 in, requiring 61 bricks to the square yard, on edge, and weigh 7 lb
RESSED bricks are 2½ by 4 by 8½ in, requiring 58 to the square yard
g 6½ lb each. METROPOLITAN bricks are 3 by 4 by 9 in, require 45
e yard, and weigh 9½ lb each.*

rtar Bricks.† **General Description.** The so-called SAND-LIME
originally made of lime mortar, molded in brick form and hardened
to the air. Such bricks are said to have been largely used in ancient
is claimed that remains of such materials are now in evidence and in
of preservation. It is known that they were formerly used in Europe
where other materials were not readily available, and that they have
n some localities in this country during the past thirty-five years.
knows of several houses in Haddonfield, N. J., built of such bricks,
th the exterior surfaces plastered. One of them, however, said to be
y-five years old, has not been plastered, and an inspection (1915)
ricks to be in an excellent state of preservation. Lime-mortar bricks
he absorption of carbonic-acid gas from the air. This gas enters
ation with the lime, forming carbonate of lime. The hardening proc-
several weeks' exposure under cover and the product has not virtues
commend it where other materials are available.

ne Bricks. It was discovered in Germany about 1875 that lime-
s could be hardened in a few hours under heat and pressure, and it
ater that the chemical reaction under the new process differs essen-
that just described, and that the percentage of lime can be greatly
he fundamental principles of sand-lime-brick manufacture are now
operty and only the details of the manufacture are patentable.
ricks were first made in Germany about 1880, and the more extended
development of the industry dates back in Europe to about 1888,

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Inspectors' Pocket-book, A. T. Byrne.
ed from article on Sand-Lime Bricks by Professor Thomas Nolan in the re-
of Building Construction and Superintendence, Part I, Masons' Work, by

cal reaction between the lime and a portion of the sa of lime, an insoluble and durable element, which bo of the sand together and forms a comparatively The small residue of uncombined lime combines, in with silica or with carbonic-acid gas from the air, until bricks thus become harder and stronger with age. I of sand-lime bricks, Edwin C. Eckel says:* "It ma sand-lime brick as marketed consists of (1) sand-gra work of (2) hydrous lime silicate, with probably (if some allied magnesium silicate, and (3) lime hydrat magnesia hydrates. These three elements will alway tural value of the brick will depend in large part or which the sand and the hydrates occur."

Quality of Sand-Lime Bricks. The quality of th upon the selection and treatment of the sand and t containing a large percentage of fine grains passing th 150 mesh, are preferable. Clay or kaolin are danger be present in quantities of more than 5%. The lime calcium lime, the magnesium silicates formed by impu as calcium silicates. Some manufacturers use re hydrate the lime themselves, before mixing it with th quicklime, mix it with the sand and slake it in the portant element affecting quality is the press. After ing, the bricks are very fragile and the press sho subjected to no shaking or friction after the pressure Vertical clay-brick presses have been commonly use well adapted to the purpose. The rotary table-presses

Tests of Sand-Lime Bricks. If the sand is reasona lime finely divided, and if the bricks are sound and they will stand weather-exposure well. If a brick s hour and the moisture rises more than ½ in, it is no moisture rises 2 in, its use for facings is questionable 3 in, it should not be used on outside work of any imp

and-lime bricks are produced having the following physical char-
ushing strength, average, between 2 500 and 3 000 lb per sq in, al-
pecimens have shown over 5 000 lb per sq in; modulus of rupture,
450 lb per sq in; fire-resistance, but little inferior to that of fire-
esistance, generally good; acid-resistance, superior; absorption,
% in 48 hours; rate of absorption, slower than for clay bricks;
ption for complete saturation, 14%; reduction of compressive
turation for absorption-test, average 33%.

perties of Sand-Lime Bricks. The bricks are square, straight,
e and homogeneous in composition and density. They cleave
ler the stroke of the trowel and present a weather-surface with the
of stone. They can be cut, carved or sand-blasted, are easily
and show no efflorescence. These claims are well established for
factured sand-lime bricks. It should be further stated that com-
d facings are made in the same press, the only difference being in
f the materials and in the handling of the raw bricks. It is there-
at a rational and homogeneous exterior wall-structure is possible,
and facings may be built and bonded in even courses, with
ner ornamental bonds. Some factories, however, manufactured,
r bricks and care should still be taken in selections from their out-
ntly the ordinary runs of sand-lime bricks are not as strong as the
uilding bricks and some of them are too low in their resistance to

and-Lime Bricks. The natural color is pearl-gray, varying in
he composition of the sand. Permanent colors are produced by
ineral oxides with the raw materials in quantities varying accord-
nsity of color desired; but as the oxides are foreign materials in
y affect the quality of the latter in proportion to the quantity

Enameled Bricks.[*] The terms GLAZED BRICK and ENAMELED
monly used, refer practically to the same product, and neither
is known as SALT-GLAZED BRICK. The enameled or glazed bricks
lipped or sprayed and then burned, whereas the salt-glaze is ob-
ntroduction of salt into the fire-boxes of kilns while the bricks are
Glazed or enameled bricks are generally divided into two classes:
eled bricks, which have a glaze containing the coloring matter
ithout any intermediate SLIP; (2) bricks which have a transparent
ver a white or colored slip, the slip coming between the glaze and
be glazed. The latter is the process most used in this country.
differ as to which process produces the best bricks although it
though the true enamel would not chip or peel as readily. These
nade in a variety of colors, from white to dark green or chocolate,
a HIGHLY GLAZED FINISH or in a DULL, SATIN-FINISH, the latter
ite desirable in many instances on account of its doing away with
more highly glazed bricks or tiles. An enameled surface may be
rom a glazed surface by chipping off a piece of the brick. The
ill show the layer of slip between the glaze and the body of the
he enameled brick will show no line of demarcation between the
rick and the enamel. American enameled and glazed bricks are
y used for the exterior surfaces of buildings, particularly for street-

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ption of the process of manufacture, see pages 320 to 322 in Building Con-
perintendence, *Part I, Masons' Work, by F. E. Kidder.*

fronts and light-courts, and for interior side walls and partitions
buildings used for a great variety of purposes.

Sizes of Enameled Bricks. Enameled bricks are made in two reg
English size, 9 by 3-in enameled surface, 4½-in bed, and (2) America
2¼-in enameled surface, 4⅛-in bed. The English-size bricks cost
1 000 more than the American, but on account of the saving in t
bricks, labor of laying and mortar in joints, the former really effe
about 7 cts per sq ft. Enameled bricks are made, also, with
enameled surface, 2¼-in bed.

Cost of Enameled Bricks. The selling price of enameled brick
$75 per 1 000 for the American size to $85 for the English size an
12 by 4⅛ by 2¼-in size; and at these prices the cost of the bric
foot is:

American size, 7 bricks to the foot. . . . .
English size, 5⅓ bricks to the foot. . . . .
English flat, 3⅗ bricks to the foot. . . . .
12 by 4⅛ by 2¼-in, 3 bricks to the foot. . . . .

Colors of Enameled Bricks. The standard colors carried in st
cream and buff; other colors are made to order.

Estimating Quantities and Cost of Brickwor

Methods of Calculation. The almost universal method of
cost of brickwork is by estimating the number of thousands of
MEASURE, and then multiplying by a certain price per thousand, w
determined by experience and which is intended to include every
the cost, and very often the profit. All of the common brickwor
building is usually figured at the same price per thousand bricks, t
for the more expensive portions of the work being made in the man
ing. The principle underlying this system is explained as follows

"The plain dead wall of brickwork is taken as the standard,
difficult, complicated, ornamental, or hazardous kinds of work are
to it so as to make the compensation equal. To illustrate, if, in on
can lay 2 000 bricks in a plain dead wall, and can lay only 500 in a
chimney-top in the same time, the cost of labor per thousand in
four times as much as in the dead wall, and he is entitled to extra
but instead of varying the price, the custom is to vary the measure
pensate for the difference in the time, and thus endeavor to secu
price per thousand for all descriptions of ordinary brickwork, inste
ent price for the execution of the various kinds of work." *

Measurements of Brick-Quantities. PLAIN WALLS are quite unive
at 15 bricks to the square foot of an 8 or 9-in wall, 22½ bricks pe
of a 12 or 13-in wall, 30 bricks per square foot of a 16 or 17-in
bricks for each additional 4 or 4½ in in the thickness of the wall.
are used without regard to the size of the bricks, the effect of the
taken into account in fixing the price per thousand. No deductio
OPENINGS of less than 80 sup ft, and when deductions are made for la
the width is measured 2 ft less than the actual width. HOLLOW W
measured as if solid. To the number of bricks thus obtained

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* From Rules of Measurement adopted by the Brick Contractors' Excha
Col.

r piers, chimneys, arches, etc. FOOTINGS are generally measured
l by adding the width of the projection to the height of the wall.
ings project 6 in on each side of the wall, 1 ft is added to the actual
all. CHIMNEY-BREASTS and PILASTERS are measured by multi-
of each breast or pilaster from the intersections with the wall by
then by the number of bricks corresponding with the thickness of
FLUES in chimneys are always measured solid. Detached
HIMNEY-TOPS are measured as a wall having a length equal to the
and two ends of the chimney, and a thickness equal to the width
Thus a chimney measuring 3 ft by 1 ft 4 in would be measured
a wall, 5 ft 8 in long. The rule for INDEPENDENT PIERS is to
ight of each pier by the distance around it in feet, and consider
he superficial area of a wall whose thickness is equal to the width
practice, many masons measure only one side and one end of a
. ARCHES of common bricks over openings of less than 80 sup ft
egarded in estimating. If the arch is over an opening larger than
ight of the wall is measured from the springing-line of the arch.
s made in the wall-measurement for stone sills, caps, or belt-
stone ashlar, if the same is set by the brick-mason. If the ashlar
ne-mason, the thickness of the ashlar is deducted from the thick-
l. The sum of all of these measurements represents a certain
SANDS OF BRICKS, and the whole is then multiplied by a common
nd, as $6, $8, $12, or $16, according to whatever the cost of plain
be. If the building is to be faced with PRESSED BRICKS, the actual
sed bricks, as nearly as it can be computed, is added to the esti-
he common brickwork, nothing being added for laying the pressed
thing deducted from the common-brick measurement, the meas-
common work displaced by the pressed bricks being assumed to
ence in the cost of laying the pressed and common brickwork.
the COST OF THE PRESSED BRICKS, the external superficial area of
with such bricks is computed, and all openings, belt-courses, stone
educted. Five-in stone sills are not usually deducted. If a por-
is covered by a porch, so that common bricks may be used back
, also, is deducted. The net pressed-brick surface is then multi-
, or 7 to obtain the number of bricks required, 6½ giving about
pressed bricks of the standard size required to the square foot.
UT of chimneys, if of face-brick, is measured by girting the chim-
ng by the heights, and adding the sums to the wall-area.

s a simple example of this system of estimating consider a small
by 32 ft in plan, without cross-walls, the basement-walls being
h footings 2 ft 6 in wide; the first-story walls, 13 in thick; the
lls, 9 in thick; the height of the basement-walls from the trench to
first-story joists, 8 ft 6 in; the height of the walls from the first-
he top of the second-story joists, 10 ft 6 in; and from the second-
he plate, 9 ft.

UREMENTS. Basement-walls: 120 ft (girth of building) by 9 ft
d projection of footing) by 22½ bricks per square foot; equal to

alls: 120 ft by 10 ft 6 in by 22½ bricks per square foot; equal
s.

walls: 120 ft by 9 ft by 15 bricks per square foot; equal to

two chimneys, each 1 ft 9 in by *1 ft 5 in by 14* ft high above roof;

2 by 14 ft by (1 ft 5 in plus 1 ft 9 in plus 1 ft 5 in) by
equal to 3 850 bricks.

Total brickwork: 74 960 bricks. At $9 per 1 000,

PRESSED BRICKS. From the grade to the under s
measures 22 ft 6 in and it is to be faced with pressed
costing $15 per 1 000. The door-openings and wind
sup ft.

The surface of pressed bricks equals 120 by 22½ ft, e
The deduction for openings is. . . . .

Area, after deduction. . . . .
Addition for two chimneys, 2 by 14 by 6 ft 4 in, equa

Total

2 493 by 6½ equals 16 204 pressed bricks, which, at
$243.

The total amount of the bid is $674.64 plus $243, o

The above figures are supposed to include the ne
scaffolding, etc., required to make the mortar and pu
profit for the contractor; but anything in the way
thimbles, ash-doors, etc., are figured as additions to t

Detailed Estimates of Brickwork. In estimating
price per thousand is to some extent a matter of gues
rienced contractor may perhaps make as accurate an
is possible by any, yet it is often necessary to estima
even when the work has been estimated as above, it
tractor to know how many bricks and how much sand
to do the work. The following data will assist in maki

With the size of bricks used in the Western States,
bricks are required to the cubic foot after deducting
thickness of walls at 8, 12, 16, 20 in, etc., the actual
will run about two-thirds of the WALL-MEASURE when
the average number and size.

The number of pressed bricks will be about 6 or 6
deducting openings.

To lay 1 000 common bricks, kiln-count, requires 2
lime and 5/8 cu yd of sand. For a good lime-and-ceme
of lime, 1 bbl of cement and 5/8 cu yd of sand.
mortar, allow 1½ bbl of cement and 5/8 cu yd of sand,

To lay 1 000 pressed bricks with buttered joints wi
(160 lb) and ¼ cu yd of sand; with spread joints, fro
and from 3/8 to ½ cu yd of sand.

If colored mortar is used, about $1 per 1 000 brick
mortar-color.

A brick-mason, working on a city job under a goo n brickwork, one helper will be required for every mason, and on ed with pressed bricks, one helper to every two masons. In build- rick fireplaces and chimneys one mason and helper will lay about a day of nine hours.

himneys built of common bricks and with 4-in walls cost about 50 ng foot, in height, for single flues, and 90 cts for double flues.

quired for Piling Bricks. One thousand bricks closely stacked 56 cu ft of space. One thousand old bricks, cleaned and loosely py about 72 cu ft.

er's hod measures 21 by 7 by 7 in, and will hold 18 bricks.

od measures 24 by 12 by 12, and 12 in across the top.

lors are usually in the form of dry powders, or of pulp or paste. are put up in barrels, the number of pounds to the barrel and price ing about as follows:

b barrels, dry	from 1¾ to 2 cts per lb
-lb barrels, dry	from 1¾ to 2½ cts per lb
b barrels, dry	from 1¾ to 2½ cts per lb
o-lb barrels, dry	from 3 to 3½ cts per lb

less than full barrels an extra charge is sometimes made for packing

paste-form:

nd buff	1¾ cts per lb
	3 cts per lb
rs	2 cts per lb

aste-form can be obtained in casks, barrels, half-barrels and kegs, ack and buff) weighing, in casks, 900 lb; in barrels, 550 lb; and in 75 lb. The buff weighs, in casks, 700 lb; in barrels, 450 lb; and in oo lb. Black weighs, in barrels, 450 lb; and in half-barrels, 275 lb. mortar for laying 1 000 bricks with ⅛-in joints requires about 50 a-cotta color, amber, fern-green and salmon; 40 lb for buff, brown, or French gray; and 25 lb for black. For wider joints, a larger stain must be used. For paste-colors an average mixture is, 1 ste-color to 7 buckets of mortar for brickwork with ⅛-in joints. lors are in the form of dry powder they are first mixed with dry l slaked lime is then added and again mixed thoroughly. It is very at the color be uniformly mixed. If it is not added at first, but left tar is made, the labor of mixing is doubled. The more thorough e less color is required. Mortar colors should never be mixed with hen the color is in the form of a pulp or paste, it should be thor- in, in order to secure a uniform and smooth shade. For very fine s, the stained mortar should be strained through a coarse sieve.

nce on Brickwork. A white EFFLORESCENCE often appears on pecially in moist climates and damp places. It may spread over the wall-surface although originating in the mortar joints. Solu- cipally of soda, potash and magnesia, in the cement or lime mortar, by the water absorbed by the mortar and later precipitated on the e brickwork as a white deposit, when the water evaporates. This to be greater with the natural than with the Portland-cement mor- heavier with lime mortar. The origin of the efflorescence may be in emselves as well as in the mortar used. This is the case when the ade from clays containing iron pyrites or burned with sulphurous

coal. Moisture in such bricks tends to dissolve the sulphate
sulphate of lime, which, in the evaporation of the water, are
surface as crystals of these salts. Efflorescence may result,
impregnated from the mortar, absorbed by the bricks and
leaving the whitish deposits; and it is sometimes caused by
certain MORTAR-COLORS. As a PREVENTIVE, General Gilmore
addition to every 300 lb of the cement powder, 100 lb of quick
to 12 lb of any cheap ANIMAL FAT, which is to be thoroughly
the quicklime before the latter is slaked, preparatory to adding
The alkaline salts tend to be SAPONIFIED by the fat. This
satisfactory treatment, and as a rule it only partly prevent
objectionable deposits; and this addition to the cement retar
somewhat diminishes its strength. It is claimed by some tha
OIL, applied to brickwork in two coats, will lessen the absorpti
from one to three years and thus lessen the tendency to ef
usually mixed in the proportion of 2 gal of oil to 300 lb of dry
or without lime; but it is injured by the mortar and, like th
setting of the cement mortar and weakens it. In order to di
of efflorescence on brickwork, the walls should be made as IMPE
by laying the bricks in a rich well-mixed Portland-cement mo
joints full and solid. If the building is on damp ground, ca
DAMP-PROOF COURSES of the proper materials should be built
course of horizontal joints near the bottom of the walls should
Reasonably hard bricks should be used for facing, projection
surfaces waterproofed and provided with drips, and the roof,
made water-tight. When efflorescence is due to the penetrati
moisture into the brickwork and it is required to preserve the
of the work, the surface may be coated with preparations of
various patented WATERPROOFING MIXTURES. The prepa
paraffine are usually applied hot, and the walls, also, are
heaters previous to the application. They give fairly goo
quite expensive, owing to the time and labor required for
Brick walls may be rendered impervious to moisture by was
SYLVESTER PROCESS. These washes consist of an ALUM-SOLU
solving 1 lb of alum per gallon of water, and a SOAP-SOLUTION
2½ lb of pure hard soap per gallon of water. The brick walls
clean and it is recommended that they should not be colder
soap-wash is made boiling hot and then applied to the bric
perature of the alum-solution is usually from 60° to 70° F. w
wash is applied and allowed to dry for about 24 hours, after wh
is put over it. When ALUMINIUM SULPHATE, improperly calle
tuted for the alum, the cost of the wash is less, only two-thirds
as alum is required and the results are better.

LIME[*]

Nature and Properties of Lime. Chemically, lime is ca
in a broader sense, it is the class-name of a great variety
factured by the calcination of LIMESTONE. Limestone consist
of calcium and magnesium which vary widely in their ratio t
limestones used in the manufacture of lime products may b

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[*] Valuable practical data relating to lime and plaster has been furn
Warner Company, of Wilmington, Del.

LCIUM LIMESTONES and DOLOMITIC LIMESTONES. High-calcium lime-
tain only a relatively low percentage of magnesium carbonate, while
imestones contain a considerable amount of it. Dolomitic limestone
rresponds roughly to the theoretical formula of dolomite (CaCO$_3$)
The CALCINATION of limestone consists of heating to expel the carbon
The product resulting from calcination of limestone is known as
and possesses great affinity for water. SLAKING is the process of
ter to quicklime. During the process of slaking, heat is energetically
d much of the water driven off in the form of steam. During this
cess, also, high-calcium quicklimes must be agitated and stirred con-
a portion will fail to receive the proper quantity of water and will
slaked particles which are likely to slake after being used in the work,
PPING, PITTING and disintegration. Dolomitic limes do not slake so
ly, and while they should be stirred while slaking, this is not so neces-
h high-calcium limes. Either class of quicklime, through faulty manu-
likely to contain over-burned portions which slake with difficulty and
popping, etc., if the lime-paste is not carefully screened before use.
NG and HARDENING of common lime mortar is due, first, to the drying
condly, to the absorption of carbon dioxide from the atmosphere and
ion of crystals of calcium carbonate to which the strength of the mor-
bed. In the manufacture and use of common lime mortar, therefore,
terial, limestone, is first calcined, and the carbon dioxide expelled; it
ked with water and forms calcium hydroxide, in which the water is
replaced by carbon dioxide. The lime thus eventually returns to its
rbonate form. As far as the ultimate result is concerned, there is
little difference between high-calcium and dolomitic quicklimes.
reater familiarity with one or the other of the classes of lime, archi-
builders in certain sections of the country prefer one to the other.

ations for Quicklime. The lime industry has in recent years been
subject of careful study and the following clauses give the essential
tentative specifications for quicklime suggested by the American So-
esting Materials as embodying results approved in general by the best

I. Definitions, Classification, etc.

NITION. Quicklime is a material the major part of which is calcium
lcium and magnesium oxides, which will slake on the addition of water.
DES. Quicklime is divided into two grades:
cted. A well-burned lime, picked free from ashes, core, clinker or
gn material.
-of-Kiln. A well-burned lime without selection.
TS. Quicklime is shipped in two forms:
p Lime. The size in which it comes from the kiln.
erized Lime. Lump lime reduced in size to pass a ¼-in screen.
S OF PURCHASE. The particular grade, form and class shall be speci-
ance of purchase.

II. Chemical Properties and Tests

(A) Sampling

IN BULK. When quicklime is shipped in bulk, the sample shall be so
it will represent an average of all parts of the shipment from top to
d shall not contain a disproportionate share of the top and bottom

composition into four types:
(*a*) High-Calcium. Quicklime containing over 90
(*b*) Calcium. Quicklime containing not under 8
calcium oxide.
(*c*) Magnesium. Quicklime containing between
oxide.
(*d*) Dolomitic. Quicklime containing over 25% o
9. CHEMICAL PROPERTIES. (*a*) Selected quicklim
90% of calcium and magnesium oxides and not over
(*b*) Run-of-kiln quicklime shall contain not under
nesium oxides, and not over 5% of carbon dioxide.

III. Physical Properties and

10. PERCENTAGE OF WASTE. An average 5-lb sa
and slaked by an experienced operator with suffic
maximum quantity of lime putty, care being taken
ing the lime. It shall be allowed to stand for 2
through a 20-mesh sieve by a stream of water ha
No material shall be rubbed through the screens.
of the selected quicklime nor over 5% of the weight
shall be retained on the sieve. The sample of lum
shall be broken so that all of it will pass a 1-in screen
screen. Pulverized lime shall be tested as received.

IV. Inspection and Rejectio

11. INSPECTION. (*a*) All quicklime shall be subje
(*b*) The quicklime may be inspected either at the
point of delivery, as arranged at time of purchase.
(*c*) The inspector representing the purchaser shall
while work on the contract of the purchaser is being
the manufacturer's works which concern the man
ordered. The manufacturer shall afford the inspect

ECTION. Unless otherwise specified, any rejection based on failure
ts prescribed in accordance with these specifications shall be reported
days from the taking of samples.

EARING. Samples which represent rejected quicklime, shall be pre-
ir-tight containers for five days from the date of the test-report. In
atisfaction with the results of the tests, the manufacturer may make
rehearing within that time.

d Lime. The slaking of quicklime is an operation which is almost
carried on by laborers who have little or no conception of the impor-
eir task. As a result, many failures have been charged to lime in the
actually were due to improper preparation during the slaking opera-
new product known as HYDRATED LIME has been offered widely to the
cent years and has met with much success. Hydrated lime is a dry
owder resulting from the slaking of quicklime by mechanical means,
ount of water which is sufficient to satisfy the calcium oxide, but in-
make a paste or putty. Hydrated lime is manufactured in mechani-
ors in which the batches of quicklime and water used are carefully
ed by weight. After passing from the hydrator, hydrated lime is
to a mechanical system of separation which eliminates the coarse or
ticles which may cause popping, etc. Hydrated lime is sold in bags of
ight and requires only to be mixed with sand and water to make the
he bags have usually been made of heavy burlap or duck cloth, con-
lb, or of paper, containing 40 lb. Several of the more prominent
rers of hydrated lime in the United States employ chemists who
uperintend the manufacture of hydrated lime, just as the chemists in
ement factories superintend the proportioning of the raw mix going
s to be burned for Portland cement. The hydrated lime manufac-
r such chemical supervision is a reliable product free from tendencies
ht give rise to popping, pitting or disintegration. Hydrated lime of
ty may be used for almost any purpose for which lime mortar is used,
ome considered a more reliable product than quicklime. Among the
for hydrated lime may be mentioned its employment in cement mor-
oncrete. An addition of about 15% of hydrated lime to cement
concrete decreases its premeability to water, reduces the cracking
inkage, etc., and increases the plasticity of the mortar or concrete,
nting separation of the sand, stone and cement and causing the mix-
w and fill the forms more readily.

ations for Hydrated Lime. The following clauses give the essential
tentative specifications for hydrated lime suggested by the American
Testing Materials as embodying results approved in general by the
ce.

I. Definition

NITION. Hydrated lime is a dry flocculent powder resulting from the
of quicklime.

OF PURCHASE. If a particular type of hydrated lime is desired, it
ecified in advance of purchase.

II. Chemical Properties and Tests *

LING. The sample taken from hydrated lime shall be a fair average
ment 3% of the packages shall be sampled. The sample shall be

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clauses are being revised (1915) by a committee of the Am. Soc. for Test.
may be found in the 1915 *Year-Book of that society.*

taken from the surface to the center of the package.
the laboratory shall immediately be transferred to
which the unused portion shall be stored until the hydr
accepted or rejected by the purchaser.

4. Classes. Hydrated limes are commercially di
chemical composition into four classes:

(*a*) High-Calcium. Hydrated lime, the non-volati
tains over 90% of calcium oxide.

(*b*) Calcium. Hydrated lime, the non-volatile port
under 85% and not over 90% of calcium oxide.

(*c*) Magnesian. Hydrated lime, the non-volatile p
between 10 and 25% of magnesium oxide.

(*d*) Dolomitic.* Hydrated lime, the non-volatile p
not under 25% of magnesium oxide.

5. Chemical Properties. Hydrated lime shall
carbon dioxide and sufficient water to fully hydrate t

III. Physical Properties and Te

6. Fineness. A 100-g. sample of hydrated lime sh
due of not over 5% on a standard 100-mesh sieve and
ard 30-mesh sieve.

7. Constancy of Volume. A pat about 3 in in di
center, tapering to a thin edge, made on a clean glass
from a paste composed of equal parts, by weight, of
and volume-constant Portland cement (gauged with onl
the mixture workable), shall, when exposed in any
hardening 24 hours in moist air) to steam above boilin
vessel for 5 hours, show no signs of popping, checki
disintegration.

IV. Packing and Marking

8. Packing. Hydrated lime may be packed eithe
and the weight is to be plainly marked on each packa

9. Marking. The name of the manufacturer sh
tagged on each package.

V. Inspection and Rejection

10. Inspection. (*a*) All hydrated lime shall be su

(*b*) The hydrated lime may be inspected either at
or the point of delivery, as arranged at the time of pu

(*c*) The inspector representing the purchaser shall ha
while work on the contract of the purchaser is being per
manufacturer's works which concern the manufactu
ordered. The manufacturer shall afford the inspecto
for inspection and sampling, which shall be so conduct

ECTION. Unless otherwise specified, any rejection based on failure to prescribed in these specifications shall be reported within five working the taking of samples.

IEARING. Samples which represent rejected hydrated lime shall be in air-tight containers for five days from the date of the test-report. dissatisfaction with the results of the tests, the manufacturer may n for a rehearing within that time.

me. A recent development in the lime industry is Alca Lime.* This ial said to combine the plasticity and sand-carrying qualities of lime th the strength, hardness and quicker set of the gypsum plasters. osed of approximately 85% of hydrated lime and 15% of a specially naterial containing alumina and silica in such proportions as to com- ing bodies which greatly contribute to the strength, hardness and of the product. It is sold in 100-lb packages and requires only to be h sand and water before use. When used for plastering, it has the tics of lime mortar, and while it becomes hard and strong, it is claimed ee from the so-called sounding-board effects noticed in some hard-wall It is not injured by water and is often used for outside stucco-work a brick-laying mortar in place of lime mortar gauged with Portland The manufacturers' directions for the use of Alca Lime should be care- ved, and this may be said of all prepared plastering or cementing

Data on Quicklime. Quicklime is shipped either in barrels or in bulk. nates it will keep for a long time in bulk, but in damp climates and oast it soon slakes unless enclosed in barrels. In most of the Eastern old by the barrel, weighing for Rockland Me. lime 220 lb net. When bulk it is generally sold by the bushel of 80 lb, 2½ bushels or 200 lb ng considered as equivalent to a barrel. Other weights are, 230 lb, l, 75 lb per bushel and 64 lb per cu ft. The average yield of LIME- the best Eastern limes has been found to be 2.62 times the bulk of me. A barrel of good quality well-burned lime should make 8 cu ft, or lime-paste or putty. Careful experiments conducted by United States ave demonstrated that the best mortar is obtained by mixing one part e to two parts of sand.

s. For data on cements, see Chapter III.

SAND AND GRAVEL

obtained from banks or pits, from river-beds and from the seashore. bank-sand, free from clay or earthy materials, is generally considered r mortar, although excellent sand is often obtained from river-beds. ontains alkaline salts which attract and retain moisture and which, oughly washed, cause efflorescence when used in brickwork. Both d river-sand have more or less rounded grains, to which lime or cement here as well as to sharp, angular grains. Both are extensively used, r lack of better materials. The use of sand in mortar is to prevent rinkage and to save the cost of lime or cement. Sand, when used in tion of 1 : 2, strengthens lime mortar, but any addition of sand to kens it.

g Sand. Sand for mortar must ordinarily be screened. Sand for tar for plastering or common brickwork is ordinarily run through a

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patented article and is offered for sale by many licenses in the United States ackman patents.

LATHING AND PLASTE

Wooden Laths should be well seasoned, free fron
Bark on laths is quite sure to stain the plaster. V
sidered the best wood for laths, although spruce a
used. Hard pine is not a good material, as it cont
regular size of laths is ¼ in by 1½ in by 4 ft. The wid
what in different mills. There is a new lath on the
long and which costs from $1.75 to $2 less than th
sold by the thousand, in bunches containing 100 lat
ing about the average prices.

Metal Lathing. (See Chapter XXIII, pages 88

Plastering on laths is generally done in three coat
the SCRATCH-COAT; the second, the BROWN COAT, and
SKIM-COAT, or FINISH. On brickwork or stonework t
omitted. For first-class work each coat should be p
before the next coat is applied, and under no circu
coat be applied before the brown coat is thoroughly

Drawn Work is a brown coat applied to a scratch-
immediately after the scratch-coat is applied. It i
SCRATCH, and much of it is done in the Western Sta

The Scratch-Coat should always be made rich in l
bu of hair, or an equivalent quantity of fiber to each
to 2 of lime. A proportion of one part lime-paste t
quire 1 cask (2⅗ bu) of lime to 5½ bbl of screened

The Brown Coat should contain 1 cask (2½ bu) c
sand, and 1 bu of hair to 5 of lime. Very little plaste
ever, the usual custom being to mix as much sand
mortar-mixer thinks it will stand and give satisfaction
to make the lime go as far as possible.

es is generally composed of lime-putty and washed beach-sand in
tions.

h, which has a rough surface resembling coarse sandpaper, is mixed
way, only that coarser sand and more of it is used, and it is finished
en or cork-faced float.

ting or Hard Finish generally means a composition of lime-putty
f Paris, to which marble-dust is sometimes added. Plaster of Paris
lust when used should not be mixed with the lime-putty until a few
ore using, and no more should be prepared at one time than can be
nce, as it soon SETS, after which it should not be used. The skim-
finish should be finished with a steel trowel and wet brush. The
rk is troweled the harder it becomes. A superior hard finish is ob-
ixing 4 parts of Best's Keene's cement to 1 part lime-putty.

Plastering. To make sure that the lime is well slaked, it is cus-
quire that the mortar for plastering shall be mixed at least seven
it is used.

as is used by plasterers is obtained from the hides of cattle, and after
l and dried is put up in paper bags, each bag being supposed to con-
l of hair when beaten up. Each package is supposed to weigh from
the weight often falls short. ASBESTOS and MANILLA FIBER are both
of hair; they are cleaner than hair and are said to be less injured by
t is much better to add the hair to the lime-paste AFTER IT IS COLD
ixing in the sand, as hot lime, and the steam caused by the slaking,
he hair so as to greatly weaken it. The common practice is to put
he mortar-box, run off the hot lime as soon as it is slaked, throw in
l mix the whole together. It is then thrown out of the box into a
ew batch mixed up.

-Made Mortar. In several of the larger cities plants have been
the mixing of mortar by machinery. Machine-mixed mortar should
ter than the ordinary hand-mixed mortar, for the reason that time
for the lime to slake, the lime and sand can be accurately measured,
and lime are not mixed with the lime until just before delivery. The
also be more thoroughly and evenly done by machinery than is
hand.

l Wall-Plasters. Owing to the difficulty of obtaining sufficient space
perations in central sections of large cities to properly slake sufficient
to carry on the plastering with the necessary speed, other kinds of
aterials have come into existence in recent years. These are known
PLASTERS OR HARD-WALL PLASTERS. The base of these products is
phate or gypsum which has been calcined to partially expel the
setting and hardening of these products is dependent upon their
chemically with the gauging water and crystallizing in the same
rm as the material possessed before calcination. All hard-wall
tain material added for the purpose of controlling the SET. The
ined gypsum sets in a very few minutes, which time would be en-
hort to permit the workmen to apply the plaster to the wall and
up before it had set. These plasters are characterized, also, by
ty to carry as much sand as lime mortar. Many of them contain
nces, such as clay or hydrated lime, added to improve their PLASTIC-
wall plasters manufactured in the eastern part of the United States
ypsum invariably contain 15%, more or less, of clay or hydrate,
is purpose. Plasters made in Kansas, Oklahoma, Texas and other

these materials, the gypsum is calcined, immersed i
chemical and recalcined. The name KEENE'S CEMEN
materials, which are made by several manufacturers
slow-setting and ultimately attain great strength and
is generally used with considerable lime-putty or h
equal parts of hydrated lime and Keene's cement in r
is often recommended and found in specifications.
wall-plaster, see page 1467.)

Advantages of Improved Wall-Plasters. Among
the use of these plasters are uniformity in strength
and toughness, freedom from pitting, saving in time
ing, minimum danger from frost while being applied
and moisture in the building, and, in some cases, gre
of fire.

Measuring Plasterers' Work. Lathing is alway
and is generally included with the plastering, althoug
carpenter often puts on the laths. Plastering on pl
and ceilings, is always measured by the square yard,
coat, or three-coat work, or lime or hard plaster.
openings, custom varies somewhat in different parts o
different contractors. Some plasterers allow one-ha
ordinary doors and windows, while others make no al
than 7 sq yd.

Miscellaneous Details. Returns of chimney-brea
less than 12 in in width should be measured as 12 i
stairs, etc., are generally figured at a higher rate than
is not as easy to get at them. For circular or ellipti
ceilings, an additional price is made. If the plaste
trestles an additional charge must be made for stagin
done by measurement the contract should definitely
ings are to be deducted, and a special price should be
based on the full-size details.

ıntities of Materials for Lathing and Plastering

ɔus Data. To cover 100 sq yd requires from 1 500 to 1 600 laths, r an average job, and 10 lb of threepenny fine nails.

ɔlastering on wooden laths, plaster-of-Paris finish, will require ɔu of lime, 1½ cu yd of sand, 2 bu of hair and 100 lb of plaster of q yd.

coat is omitted, deduct 2 bu of lime and all of the plaster of Paris. ıed, omit the plaster of Paris and add ½ cu yd of sand.

› sq yd with two coats on brick or stone walls, the brown coat and will require from 8 to 10 bu of lime, 1½ cu yd of sand, and 100 lb aris, to 100 sq yd.

Keene's cement for brown mortar and Keene's finish on expanded- require, for brown mortar, 550 lb of cement, 5½ bu of lime, 2 cu 2 bu of hair; for the finish, 300 lb of cement and 1 bu of lime per

rs on expanded-metal lath, plaster-of-Paris finish, require, for 2 000 lb of plaster and 2 cu yd of sand; for the finish, 1 bu of lime ɔlaster of Paris per 100 yd.

hing and Plastering. The average price for putting on wooden ıly, is 3¾ cts per yard. For expanded or sheet-metal laths on ng, 5 cts; on steel studding, wired, from 8 to 10 cts.

putting three coats on laths, plaster-of-Paris finish, labor only, cts per yard for drawn work and 16 cts for dry scratch.

nish the cost is about the same as for white finish.

s are based on plasterers' wages at 62½ cts per hour, and 37½ cts ɔd-carriers and mortar-mixers.

g schedule gives the average cost of different kinds of plastering, ıt 40 cts per bushel, sand at 75 cts per load of 1¼ cu yd, hair at 40 l, plaster of Paris at 50 cts per 100 lb, and wages as given

own coat (lime) on wooden laths	25 cts per sq yd.
me) on wooden laths, plaster-of-Paris finish	30 cts per sq yd.
me) on wooden laths, sand finish	30 cts per sq yd.
d finish on brick walls	23 cts per sq yd.
ll plaster instead of lime, add	3 cts per sq yd.
lime), plaster-of-Paris finish, metal lath on udding	65 cts per sq yd.
me) plaster-of-Paris finish, metal lath on steel	68 cts per sq yd.
cement finish, add	10 cts per sq yd.
in imitation of tile, add	50 cts per sq yd.
rd-wall plaster, plaster-of-Paris finish, metal len studding	70 cts per sq yd.
rd-wall plaster, plaster-of-Paris finish, metal eel studs	75 cts per sq yd.
cement finish, add	10 cts per sq yd.
nt, brown coat, finished with Keene's cement imitation of tile, 3 by 6 in	$2.80 per sq yd.
ase, 9 in high, in Best's Keene's cement	10 cts per ft.

lain moldings in plaster of Paris, from 3 to 5 cts per inch of girth.

hafts of columns, from 16 to 24 in in diam., from 12 to 14 ft high, umn (labor only).

These prices are believed to be pretty near an average for th
In some localities prices for materials or labor are less, in other

Staff is a composition of plaster of Paris and hemp-fiber, c
nailed or wired in place. All of the buildings of the Columbi
Chicago (1893) were covered with this material and all of the
ings of the St. Louis Exposition (1904).* It is not sufficientl
manent work unless it is frequently painted. The cost of sta
buildings at Chicago in 1893, varied from $2 to $2.25 per sq y

DATA ON LUMBER AND CARPENTERS'

Relative Hardness of Woods. Taking shell-bark hick
standard of our forest-trees, and calling that 100, other trees w
for hardness as follows:

Shell-bark hickory	100	Yellow oak	
Pignut hickory	96	Hard maple	
White oak	84	White elm	
White ash	77	Red cedar	
Dogwood	75	Wild cherry	
Scrub-oak	73	Yellow pine	
White hazel	72	Chestnut	
Apple-tree	70	Yellow poplar	
Red oak	69	Butternut	
White beech	65	White birch	
Black walnut	65	White pine	
Black birch	62		

Weight of Rough Lumber per 1 000 Feet

BOARD-MEASURE, APPROXIMATE

For weight of various woods see tables on pages 1415

Kind of wood	Green from saw, lb	Shipping-dry, lb	We seas
Ash			
Chestnut	4 600	…………	3
Hemlock	4 200	3 000	……
Maple, hard	5 400	4 150	3
Maple, soft	5 000	3 650	3
Oak, red	5 500	4 250	4
Oak, white	5 700	4 500	4
Pine, long-leaf	4 500	3 500	……
Pine, white	3 500	2 500	2
Poplar	4 000	3 000	2
Spruce	3 150	2 700	2
Sycamore	4 750	3 200	3
Walnut, black	4 900	4 000	3

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* A description of the process of manufacture of staff is given
Building Construction and Superintendence, Part I, Masons' Work,

† A comprehensive booklet giving the rules for the grading and cl
pine lumber and dressed stock may be obtained from The Yellow
Association, St. Louis, Mo.

g-Lumber may commonly be purchased in any of the following nomi- xcept that common pine, spruce, and hemlock cannot usually be ob- arger sizes than 12 by 12 in.

Nominal Sizes of Framing-Lumber

in	in	in	in
2 ×4	3× 6	4×12	8×12
2 ×6	3× 8	4×14	8×14
2 ×8	3×10	6× 6	10×10
2 ×10	3×12	6× 8	10×12
2 ×12	3×14	6×10	10×14
2 ×14	3×16	6×12	10×16
2 ×16	4× 4	6×14	12×12
2½×12	4× 6	6×16	12×14
2½×14	4× 8	8× 8	12×16
2½×16	4×10	8×10	14×14
....	....	....	14×16

of the New England mills, the following sizes, also, are sawed: 2 by 3, 7, 2 by 9, 3 by 4 and 3 by 5 in. These sizes are not commonly carried nd in most localities would have to be obtained by ripping larger st of the long-leaf yellow pine and Douglas fir is SHIPPED SURFACED ND EDGE, the actual dimensions being from ¼ in to ⅜ in, and some- , scant of the nominal dimensions. When framing-lumber is required o dimensions it should be ordered IN THE ROUGH, and a special contract hat understanding.

of Framing-Timbers. All timber is cut and sold in even lengths, as nd 16 ft. Odd and fractional lengths are counted as the next higher h; consequently it is, in certain cases, possible and economical to plan o that timbers of even lengths may be used without waste.

ement of Rough Lumber. All rough lumber is sold by the foot, SURE, one foot being the equivalent of a board 1 ft wide, 1 ft long, and To compute the board-measure in any board, plank, or timber, di- ominal sectional area, in inches, by 12, and multiply by the length in s the number of FEET in a 2 by 4-in scantling, 8 ft long = $(2 \times 4/12) \times$ board-measure. A 10-in board, 12 ft long, contains $(1 \times 10/12) \times$ board-measure. Extensive tables are published showing the feet, in sure, for almost any commercial size of timber. The following table, lthough compact, will enable one to readily estimate the number of of the standard sizes of boards, planks, or timbers. To use the table, roduct of the lateral dimensions of the cross-section; then in the ving a heading equal to this product, and in the horizontal line oppo- en length will be found the number of feet in board-measure. Thus, 4, 2 by 6, or 1 by 12-in timber look in the column headed 12; for a 2 6, or 3 by 8-in piece, look in the column headed 24. For lengths not e table, take either twice the length and divide by 2, or one-half the multiply by 2. Where timbers of the same size abut end to end, it labor in reducing to board-measure to take the full length; for this lengths in the table are carried beyond those for single sticks.

Table of Board-Measure

For explanation, see page 1473

Length in feet	Sectional area in square inches										
	4		6	8		10		12	14		1
	ft	in	ft*	ft	in	ft	in	ft *	ft	in	ft
6	2	0	3	4	0	5	0	6	7	0	8
8	2	8	4	5	4	6	8	8	9	4	10
10	3	4	5	6	8	8	4	10	11	8	13
12	4	0	6	8	0	10	0	12	14	0	16
14	4	8	7	9	4	11	8	14	16	4	18
16	5	4	8	10	8	13	4	16	18	8	21
18	6	0	9	12	0	15	0	18	21	0	24
20	6	8	10	13	4	16	8	20	23	4	26
22	7	4	11	14	8	18	4	22	25	8	29
24	8	0	12	16	0	20	0	24	28	0	32
26	8	8	13	17	4	21	8	26	30	4	34
28	9	4	14	18	8	23	4	28	32	8	37
30	10	0	15	20	0	25	0	30	35	0	40
32	10	8	16	21	4	26	8	32	37	4	42
34	11	4	17	22	8	28	4	34	39	8	45
36	12	0	18	24	0	30	0	36	42	0	48
38	12	8	19	25	4	31	8	38	44	4	50
40	13	4	20	26	8	33	4	40	46	8	53
42	14	0	21	28	0	35	0	42	49	0	56

Length in feet	Sectional area in square inches								
	24	28		30	32		35		36
	ft *	ft	in	ft *	ft	in	ft	in	ft *
6	12	14	0	15	16	0	17	6	18
8	16	18	8	20	21	4	23	4	24
10	20	23	4	25	26	8	29	2	30
12	24	28	0	30	32	0	35	0	36
14	28	32	8	35	37	4	40	10	42
16	32	37	4	40	42	8	46	8	48
18	36	42	0	45	48	0	52	6	54
20	40	46	8	50	53	4	58	4	60
22	44	51	4	55	58	8	64	2	66
24	48	56	0	60	64	0	70	0	72
26	52	60	8	65	69	4	75	10	78
28	56	65	4	70	74	8	81	8	84
30	60	70	0	75	80	0	87	6	90
32	64	74	8	80	85	4	93	4	96
34	68	79	4	85	90	8	99	2	102
36	72	84	0	90	96	0	105	0	108
38	76	88	8	95	101	4	110	10	114
40	80	93	4	100	106	8	116	8	120
42	84	98	0	105	112	0	122	6	126

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* The measurements in these columns come out in ev

Table of Board-Measure (Continued)

For explanation, see page 1473

Length in feet	Sectional area in square inches													
	56		60	64		72	80		84	96	100		112	
	ft	in	ft *	ft	in	ft *	ft	in	ft *	ft *	ft	in	ft	in
4	18	8	20	21	4	24	26	8	28	32	33	4	37	4
6	28	0	30	32	0	36	40	0	42	48	50	0	56	0
8	37	4	40	42	8	48	53	4	56	64	66	8	74	8
10	46	8	50	53	4	60	66	8	70	80	83	4	93	4
12	56	0	60	64	0	72	80	0	84	96	100	0	112	0
14	65	4	70	74	8	84	93	4	98	112	116	8	130	8
16	74	8	80	85	4	96	106	8	112	128	133	4	149	4
18	84	0	90	96	0	108	120	0	126	144	150	0	168	0
20	93	4	100	106	8	120	133	4	140	160	166	8	186	8
22	102	8	110	117	4	132	146	8	154	176	183	4	205	4
24	112	0	120	128	0	144	160	0	168	192	200	0	224	0
26	121	4	130	138	8	156	173	4	182	208	216	8	242	8
28	130	8	140	149	4	168	186	8	196	224	233	4	261	4
30	140	0	150	160	0	180	200	0	210	240	250	0	280	0
32	149	4	160	170	8	192	213	4	224	256	266	8	298	8
34	158	8	170	181	4	204	226	8	238	272	283	4	317	4
36	168	0	180	192	0	216	240	0	252	288	300	0	336	0
38	177	4	190	202	8	228	253	4	266	304	316	8	354	8
40	186	8	200	213	4	240	266	8	280	320	333	4	373	4
42	196	0	210	224	0	252	280	0	294	336	350	0	392	0
44	205	4	220	234	8	264	293	4	308	352	366	8	410	8
46	214	8	230	245	4	276	306	8	322	368	383	4	429	4
48	224	0	240	256	0	288	320	0	336	384	400	0	448	0
50	233	4	250	266	8	300	333	4	350	400	416	8	466	8
52	242	8	260	277	4	312	346	8	364	416	433	4	485	4
54	252	0	270	288	0	324	360	0	378	432	450	0	504	0
56	261	4	280	298	8	336	373	4	392	448	466	8	522	8
58	270	8	290	309	4	348	386	8	406	464	483	4	541	4
60	280	0	300	320	0	360	400	0	420	480	500	0	560	0
62	289	4	310	330	8	372	413	4	434	496	516	8	578	8
64	298	8	320	341	4	384	426	8	448	512	533	4	597	4
66	308	0	330	352	0	396	440	0	462	528	550	0	616	0
68	317	4	340	362	8	408	453	4	476	544	566	8	634	8
70	326	8	350	373	4	420	466	8	490	560	583	4	653	4
72	336	0	360	384	0	432	480	0	504	576	600	0	672	0
74	345	4	370	394	8	444	493	4	518	592	616	8	690	8
76	354	8	380	405	4	456	506	8	532	608	633	4	709	4
78	364	0	390	416	0	468	520	0	546	624	650	0	728	0
80	373	4	400	426	8	480	533	4	560	640	666	8	746	8
82	382	8	410	437	4	492	546	8	574	656	683	4	765	4
84	392	0	420	448	0	504	560	0	588	672	700	0	784	0

* The measurements in these columns *come out in even feet.*

Table

F

Length in feet	120 10×12 ft *	140 10×14 ft	in
4	40	46	8
6	60	70	0
8	80	93	4
10	100	116	8
12	120	140	0
14	140	163	4
16	160	186	8
18	180	210	0
20	200	233	4
22	220	256	8
24	240	280	0
26	260	303	4
28	280	326	8
30	300	350	0
32	320	373	4
34	340	396	8
36	360	420	0
38	380	443	4
40	400	466	8
42	420	490	0
44	440	513	4
46	460	536	8
48	480	560	0
50	500	583	4
52	520	606	8
54	540	630	0
56	560	653	4
58	580	676	8
60	600	700	0
62	620	723	4
64	640	746	8
66	660	770	0
68	680	793	4
70	700	816	8
72	720	840	0
74	740	863	4
76	760	886	8
78	780	910	0
80	800	933	4
82	820	956	8
84	840	980	0

* The measurem

ment of Finishing-Lumber, Flooring, Ceiling, Etc. Most, if not
or finishing is sawed for use in thicknesses of 1 in, 1¼ in, 1½ in, and
ne woods, such as white pine and poplar, are sawed into thicknesses
d 3 in.
aced both sides, the thickness is reduced to ¹³⁄₁₆, 1¹⁄₁₆, 1⁵⁄₁₆, 1¾, 2¼,
.

d stock is measured and sold STRIP-COUNT, that is, full size of rough
essarily used in its manufacture. Thus 1¹⁄₁₆-in boards are measured
¼ in thick. The number of feet, board-measure, for 1¼-in stock
d) is 1¼ times that in a 1-in board, and in the same way for 1½-in
tock. 1¾-in planks are always measured 2 in thick, and 2¼-in
thick. Boards less than 1 in thick are measured the same as 1-in
for ⅜-in and ⅝-in stock a reduced price is generally made.

Ordinary Flooring.* The standard sizes for flooring (other than
arqueting or parquet-flooring) are 1 by 3, 1 by 4 and 1 by 6; or 1¼ by
nd 1¼ by 6. The thickness of 1-in flooring should be ¹³⁄₁₆ in, and
ring, 1³⁄₃₂ in. 3-in flooring should show 2¼ in on the face, after it is
¼ in; and 6-in, 5¼ in.

Maple Flooring is usually made in 2-in, 2¼-in and 3¼-in face, and
s of ¹³⁄₁₆, 1¹⁄₁₆ and 1⁵⁄₁₆ in.

atched and beaded boards, is regularly stuck in the same widths as
he standard (nominal) thicknesses of yellow-pine ceiling are ⅜, ½,
the actual thickness of each being ¹⁄₁₆ in less. The ⅜-in ceiling is
side only, the other thicknesses both sides.

ne Drop-Siding. Dressed and matched yellow pine drop-siding is
d ¾ by 5½ in, showing 3¼ and 5¼-in face; and worked shiplap is
d ¾ by 5½ in, showing 3 and 5-in face.

iding is resawed on a bevel from stock ¹³⁄₁₆ by 3½ and ¹³⁄₁₆ by 5½ in,
ng.

and Clapboards are 4 ft long, 6 in wide, ½ in thick at the butt, and
hick at the other edge. They are put up in bunches and sold by the

Estimating Quantities of Sheathing, Flooring, Etc. For com-
ng laid horizontally on a wall or roof without openings, add one-
actual superficial area to allow for waste. On the walls of dwellings,
alls as though without openings and allow nothing for waste. If
laid diagonally, add one-sixth to the actual superficial area.
heathing laid horizontally, add one-fifth for 6-in boards, one-seventh
rds, and one-ninth for 10-in boards. If laid diagonally add one-
in boards, one-sixth for 8-in boards, and one-eighth for 10-in boards.
matched flooring add one-half to the actual superficial area to be

looring add one-third and for 6-in flooring add one-fifth. Ceiling is
e same as flooring.
siding, add one-fifth to the superficial area.
ding, laid 4 in to the weather, add one-half to the actual superficial
in to the weather, add one-third.

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ere except in New England FLOORING is always understood to be tongued and

Cost of Labor for Carpenters' Work.* There are so man
ditions which enter into the cost of carpenters' work, and the cos
with the locality, that it is quite impossible to give figures whi
practical value, although several books † have been publishe
labor and materials for buildings.

The following figures of the cost, for labor and nails, of fra
on sheathing and siding and laying flooring are probably a f
carpenters' wages at $3 a day of eight hours (37½ cts per ho
framing is almost always figured at a certain price per thousa
board-measure. The cost of laying flooring, sheathing, etc.,
figured by the square of 100 sq ft (10 by 10 ft).

Character of work	
For setting up studding and framing walls of wooden dwellings	$
For framing and setting floor-joists, 2 by 8 to 2 by 12	$9.00
Framing and setting heavy joists and girders, 6 by 12 to 10 by 14	$
Framing gable roofs and setting in place	1
Framing hip-roofs and setting in place	$11.00
For putting in bridging, after it is cut, per 100 lin ft in the row	
For covering the sides or roofs of wooden buildings with dressed sheathing, laid horizontally	60
The same, if laid diagonally	75
The cost of labor and nails for laying 6-in flooring, blind-nailed to every joist, without dressing after laying, is about	$2
For 4-in flooring, not dressed, allow	2
For 3-in flooring, not dressed, allow	2
For 3-in hard-pine flooring, hand-smoothed or traversed	3
For 3-in red-oak flooring, hand-smoothed or traversed	6
For 3-in white-oak flooring, hand-smoothed or traversed	8
For 3-in maple flooring, hand-smoothed or traversed	$10.0

BUILDING PAPERS, BUILDING FELTS AN

Sheathing-Papers,‡ Felts, Quilts, Etc. It is well known
ings when merely sheathed and clapboarded or shingled on the o
lathed and plastered on the inside, are almost sure to be hot in
in winter; and as the wood almost always shrinks, cracks resu
the wind finds its way. For these reasons some extra provisio
for keeping out the wind and the heat and cold; and it is genera

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* The wages of carpenters vary (1914) in the United States from 30 t
or from $2.40 to $4.50 per day of 8 hours. For rates per day differen
the figures showing the costs in the schedule must be changed propor

† The best and the most reliable of these that the author and editors
New Building Estimator, by William Arthur, Handbook of Cost Dat
and the Estimators' Price Book, by I. P. Hicks, to all of which archite
referred for detailed information and valuable data on costs of labor

‡ The terms BUILDING PAPER and SHEATHING-PAPER are by the pub
applied to all kinds of papers used in connection with building-construc
however, the term BUILDING PAPER is confined to the rosin-sized and
paper, while the heavier and better grades are classed as SHEATHING-

aterial that will do this so well and at so small an expense as good ers or sheathing-felts. The papers made for this purpose are com- as SHEATHING-PAPERS or BUILDING PAPERS. There is a great eathing-papers manufactured, many of them of great excellence, best are comparatively inexpensive, costing only about $1.00 per that only the better qualities of any kind of felt or paper should be here the cost of the sheathing-paper on an ordinary house is only it is poor economy to use a cheap paper, as the labor of applying it t item and the poorer the paper the more difficult the work of put- The qualities which good sheathing-paper should possess are per- enetrability to air and water and sufficient strength to permit of out tearing. Protection or proof against vermin and insects is rtant requirement. It should not be brittle nor have a lasting d, for the convenience of the builder, should be clean for handling. any papers possessing all or most of these qualities that it is deemed mention particular brands. The architect should decide for him- samples with which he has probably been furnished, what papers ted to the particular conditions; and he should then specify those , also, the manufacturers' names, instead of leaving the choice to ho will be quite sure to be guided by price rather than by quality. to tarred or saturated sheathing-papers and felts because of their ecome brittle and because they emit a strong odor and are some- able to handle. On the other hand, the advocates of tarred felts eir cheapness, warmth and even their odor, which makes them The odor gradually disappears after the clapboards, siding or ut on and the inside walls finished. Sheathing-paper is usually revious to putting on the clapboards, siding, or shingles. It is ed horizontally and should lap about 2 in over each sheet and over iously placed around the window and door-frames. If sheathing- ar material is to be placed under the clapboards or siding, laths led vertically over it, opposite each stud, and the siding or clap- to the laths; otherwise it will be difficult to put them on evenly, hickness and elastic quality of the QUILT. Shingles, however, may irectly over it. Sheathing-quilt possesses marked fire-resisting The sheathing-paper and the labor of putting it on should be in- carpenter's specifications.

d Building-Papers. These are the common grades of building re not water-proof, and should not be used on roofs or on walls in s. In dry places they protect from dust, draughts, and to some eat and cold. They are generally either a dull red or gray in color, mooth surface, and are clean to handle. They are always put up wide and usually contain 500 sq ft. The weight varies from 18 to oll of 500 sq ft, and the cost from 50 cts to $1.50 per roll.

and Deadening-Quilts. Among the insulating and deadening- n use are those mentioned below. There are also other good ma- line which are manufactured and used for insulating and deadening

-Quilt.* This consists of a felted matting of eel-grass held in place ayers of strong Manila paper by quilting. "The long, flat fibers oss each other at every angle and form within each layer of quilt minute dead-air spaces, that make a soft, elastic cushion. This

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* Made by *Samuel Cabot (Inc.), Boston, Mass.*

gether. The hair is chemically treated, so that it
makes the finished material vermin-proof and odorle

Mineral-Wool Deadeners, which are fire-proof
rock-fiber wool stitched between two sheets of bui
paper according to the grade desired, are made by th
Winona, Minn., and other firms. This company r
Lith and Feltlino, which are sound-deadening mate
manufacture, also, Linofelt, a building-quilt of fl
threads), stitched between water-proof paper or asbes
It is ¼ in thick. Linofelt for sheathing in place of o
from 1 to 1½% to the cost of a house.

Felt-Papers. There are a great many felt-paper
are made fire-proof by means of chemicals. As a rule
Cabot's QUILT, although the saving in an ordinary re
and even among the felts themselves there is quit
choosing a felt-paper for lining, the architect should
elastic enough to form a cushion, and the thicker t
above qualities, the greater will be its non-conduct
water-proof by an asphalt center, which is an advan
but some authorities think that it is doubtful if such
sound as well as felts without the asphalt center.
acoustical experts seems to show that one of the best
a combination of heavy hair-felt or felt-paper with
Two layers of felt, each from ½ to 1 in thick, are plac
layer of galvanized iron, the latter resting freely bet
form of construction is to be preferred where the de
tached to the enclosing woodwork. An additional
increases the effectiveness of the combination.

Saturated Felts.‡ Common roofing-felts are m
dry felt with coal-tar pitch. Roofing-felts are comm
15, and 20 lb to the 100 sq ft. Nothing lighter tha
roofing. They are usually sold by weight and the a

lly adapted for slaters' use, as they will carry a chalk line and le. The rolls are 36 in wide, contain 500 sq ft and weigh about

ilding Felts are usually made about 6, 10, 14 and 16 lb to the gh different manufacturers make different weights. They come de and are sold by weight.

ening Felts. These deadening-felts are made by various manu-ne of these felts * the material itself is rather hard and thin, but ich a way as to form small indentations or air-cells. This makes eaks up the sound-waves. The cost of this material in Boston er 500 sq ft.

eathing. Sheathing-papers or building felts, made of asbestos, onsiderable extent for floor-linings and for covering the outside buildings, principally on account of their fire-proof and vermin- These papers are well known in the trade and can be procured ty. They are supplied by the manufacturers in 50 or 100-lb , on a basis of the following scale of weights:

lb to the 100 sq ft
lb to the 100 sq ft
lb to the 100 sq ft
lb to the 100 sq ft
lb to the 100 sq ft
lb to the 100 sq ft
lb to the 100 sq ft
18 lb to the 100 sq ft
20 lb to the 100 sq ft
24 lb to the 100 sq ft
32 lb to the 100 sq ft
1/16 in thick
3/32 in thick
1/8 in thick

in the 1/16, 3/32 and 1/8-in thicknesses is used only for special pur- unusually thick lining is desired for possible fire-protection flues, for chimney-breasts, etc. When the weight of paper ex-he square foot it is known as ROLL-BOARD and is no longer classed o sq ft, but by thickness. For floor-linings, 16-lb paper is gen-l, this weight being sufficiently thick and strong to resist ordi- application and in handling. Asbestos felts and building papers approximately the same effect in retarding the passage of sound- felt-papers of a relatively similar thickness and quality, while nd vermin-proof qualities are a distinct advantage. The cost of nd building-felt, while somewhat greater than that of the ordi-d for similar purposes, is not excessive. The market price varies cts per lb, depending on the fluctuations of the market. For st of 100 sq ft of 16-lb asbestos paper varies from 32 to 40 cts, e market.

Papers. Neponset † Black Sheathing is water-proof and air- and clean to handle, and is an excellent paper under siding, or tin. The rolls are 36 in wide, containing 250 and 500 sq ft; about $2.25 per roll of 500 sq ft.

ed Rope Sheathing and Roofing. This is made of rope-stock, th and flexibility, and is absolutely water-proof and air-tight. It st sheathing-papers and makes a good cheap roofing for sheds, etc. The rolls are 36 in wide, containing 100, 250 and 500 sq ft about $5.00 per 500 sq ft.

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rian Sound-Deadening Felt, made by F. W. Bird & Son, East Walpole,

e approximate and vary with *locality and condition of the market.*

Parchment Water-Proof Sheathing. There are various par ings on the market which are semitransparent, have smooth su odorless, water-proof, air-proof and vermin-proof. They are ada sheathing purposes. In general 1-ply weighs 25 lb to 900 sq ft; 500 sq ft; 3-ply, 25 lb to 275 sq ft. They are 36 in wide.

Cost of Building and Sheathing-Papers in Place.* Th though necessarily restricted to a few lines, will give a general id different kinds and grades of sheathing-papers, the prices given ages for the materials APPLIED to an outside wall or roof:

Common tarred felts (15 lb per square) ....
Red rosin-sized sheathing, best grades..................
Monahan's parchment-sheathing, single-ply..............
Monahan's parchment-sheathing, double-ply..............
Monahan's ship-rigging tar-sheathing, 2-ply..............
"Neponset" black (water-proof) paper....................
"Neponset" red-rope roofing..........................
Sheathing-papers with asphalt center....................
Asbestos building or sheathing-felt, 10 lb per square........
Asbestos building or sheathing-felt, 14 lb per square........
Cabot's sheathing-quilt, single-ply......................
Cabot's sheathing-quilt, double-ply......................
Barrett's specification-felt............................
Barrett's DEFENDER, felt-sheathing......................
Sackett's water-proof sheathing........................
Empire parchment-sheathing, 1-ply......................
Empire parchment-sheathing, 2-ply......................
Empire parchment-sheathing, 3-ply......................
Barrett's red rope....................................
Barrett's black, water-proof sheathing...................

PAINT AND VARNISH †

Pigments and Vehicles. The solid ingredient of a paint is cal and is a fine powder, nearly all of which will pass through a br 100 meshes to the linear inch; in fact, most pigments are much and those formed as precipitates by chemical processes are so f no way to measure them. The liquid part is called the VEHICL ally linseed-oil, sometimes with the addition of a little turpentine solvent. In the enamel paints it is varnish and in kalsomine water paints it is a solution of glue, casein, albumen, or some si material. The cementing material is sometimes called the BIND

Ingredients of Oil-Paint. White lead and white zinc are th pigments. There are white pigments of variable composition c and zinc lead, furnace-products, composed of zinc oxide and There is also a basic lead sulphate, commercially called subli which is a similar furnace-product consisting chiefly of su These composite white pigments are largely used in mixed pai is a mixture of sulphide of zinc and sulphate of barium. It is

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* All prices are approximate and vary with locality and condition o

† The editor is indebted to Professor Alvah H. Sabin for valuable revision of the data relating to this subject.

and largely used as the basis of flat wall-finishes for interior work,
urable for exterior work. It is discolored (grey) by strong light, but
a very serious practical objection. White lead is used everywhere,
yellow somewhat in the dark. White zinc is chiefly used on interior
the whitest paint known. Both are often mixed and both are used
ints. Yellow paint is commonly chromate of lead, or chrome yellow;
ome green, which is a mixture of chrome yellow and Prussian blue;
marine, or sometimes Prussian blue. The brilliant reds are coal-tar
rule; the dull reds and browns are oxides of iron. Ochres are dull
rbon forms the base of all black paints, either as lampblack, drop-
black), or graphite. Linseed-oil is either raw or boiled. Raw oil is
s natural state as it is extracted from the seed; it should be settled
perfectly clear; it is yellow or greenish yellow in color. Boiled oil is
h has been heated to 400° or 500° F. with compounds (usually oxides)
manganese; it is darker in color than raw oil, and dries quicker.
osed in a thin film to the air is converted in about five days into a
ery substance; boiled oil undergoes this change in from 10 to 24 hours.

These are compounds of lead and manganese, dissolved in oil, and
thinned with turpentine or benzine. They act as carriers of oxygen
e air and the oil, and their addition to a paint makes it dry more
ome driers are also called JAPANS. Not more than 10% by volume of
e liquid driers should be added to oil. Excess of drier causes the
k durability. Cheap driers often contain rosin. It is well to specify
and japans should be free from ROSIN (not resin, as varnish-resins are
ome of the best driers).

Coat. This is the first coat applied to the clean surface. A priming
od is chiefly oil, and is usually equivalent to a gallon of ordinary paint
h a gallon of raw linseed-oil. Paint, however, is not thinned to make
oat for structural metal. In all wood-work, nail-holes and other de-
led with putty after the priming coat has been applied; but if the
inous, knots and resinous places must be covered with shellac varnish
priming coat is put on. Pitchy woods, such as southern yellow pine
, do not readily absorb oil, and turpentine should be substituted for
oil. Red lead is successfully used as a primer (2 parts to 1 of white
h woods; this is the standard practice in England, and is better than
ll white lead.

Painting. The priming coat having largely been absorbed by the
ond and third coat of paint are to be applied. The most common
on houses is white lead. This is commonly sold as paste white lead,
8% of oil; 100 lb of this is equal to 2.8 gal in volume, and is commonly
3½ gal of raw linseed-oil, 1 qt of turpentine and 1 pt of drier to make
aint for the second coat; or with 4 gal of oil, 1 pt of turpentine and 1
for the finishing coat. If white zinc is used, 9½ lb of dry zinc oxide
of oil make 1 gal of paint; to this, turpentine and drier should also
White lead, after about a year, begins to CHALK, that is, its surface
y and chalky; this does not indicate failure, however, and it makes a
e for repainting. Finely reticulated checking, not extending through
curs later, and when sufficiently marked indicates need of repainting.
nt, when cracks begin to extend through to the wood, repainting is
these cracks occur sooner on pitchy woods. White zinc, if used alone
(not inside) work, is very hard and tends to peel off. MIXED PAINTS
proprietary paints) generally contain zinc mixed with either white
e of the pigments based on basic lead *sulphate, and some* auxiliary

green are less durable colors than yellow, gray, or da to the fact that the chemical rays of light penetrate t gallon of paint will cover from 400 to 600 sq ft of su character of the surface. ROOF-PAINTS should contai and a smaller amount of drier or none at all. Thre roofs and galvanized-iron work should be thoroughl before painting. The shingles on the walls and roof stained with creosote stain, which consists of a pigm or some similar liquid. The creosote has some prese

Inside Painting. Door-frames and window-fram coat of paint in the shop; if they are to be finished i applied to the back only. As has already been said, any resinous knots should be varnished with shellac. are to be painted should be puttied after the priming be applied with a wooden spatula, not a steel one, to The paint for the second coat should contain as muc its vehicle should be half oil and half turpentine. T the paint dry with a dull instead of a glossy surfac painter's term. To this the next coat will adhere we coat, it may be an ordinary oil-paint. When thorou removed by lightly rubbing it with pumice and water of pigment with varnish as a vehicle. It is harder an oil-paint. It is also more expensive. It is usual to which case the last coat of oil-paint should be lightl hard and dry. A coat of enamel paint is then put on, be sandpapered or rubbed with curled hair. The final on and it may be rubbed in a like manner if a flat surf left with the gloss. It is also common practice for enamel finish by adding varnish to white lead or whit used in this case. The best varnish for this purpo a thoroughly reliable maker. The quicker-drying ALLIGATOR.

Varnish. There are two principal kinds of

iller. Liquid fillers are not desirable, as they form a poor base for
ork. A paste-filler is really a sort of paint, the pigment being silex,
rtz, and the vehicle is a quick-drying varnish made thin with tur-
zine. This is rubbed strongly in on the grain of the wood with a
h, and as soon as it has set, usually within half an hour, it is rubbed
h cloth or a handful of excelsior, the rubbing being hard across the
ood. If it is desired to stain the wood, the oil-stain may be mixed
but if a close-grained wood is used, which needs no filler, the oil-
hinned to the desired color with turpentine or benzine and applied
cleaning the filler out of moldings, corners, etc., a suitably shaped
a steel implement, may be used. If any puttying is necessary it is
fter two days the first coat of varnish is applied; after five days it
bed with curled hair or fine sandpaper to remove the gloss, so that
will adhere well; then one, two or three more coats of varnish are
ys or more apart, each coat being rubbed. The last coat may
left with the natural gloss. Outside doors, window-sills, jambs,
and all surfaces exposed to the direct rays of the sun, should be
spar-varnish and left glossy. If shellac varnish is used as the in-
is applied in the same way, but at least six coats should be applied.
are to be varnished should be treated as has been described; but
be waxed they should receive one or two coats of shellac varnish,
coats of wax, at intervals of a week, each coat being well polished
ed floor-brush made for the purpose. Floor-wax is not beeswax,
ound wax made for the purpose. Shellac is a good floor-varnish;
e wood less than any other varnish, and dries rapidly.

lastered Walls. Plastered walls which must be painted are usu-
ith a solution of soap and then with a solution of alum. When
sponged off, then allowed to dry, then oiled, then painted. If the
d to the fresh plaster the lime in the plaster will attack the paint.

. The exterior woodwork of a house needs repainting once in five
cording to climate and other conditions, although if not done with
al or sufficient care it will not last as long as this; the interior
ood care, stand from fifteen to twenty years, and then may not
ete renewal. Exterior paint sometimes loses its luster, while the
int is still good, and in cases of this kind it is sufficient to wash the
en give it a coat of oil. This replaces the oil which has superficially
rts a gloss and brings out the color. If the paint is worn off so
e wood in places, or is peeling, it must be very carefully examined.
ses it is necessary to BURN OFF the old paint; this is done with a
h, a lamp which burns alcohol, naphtha, or kerosene, and which
ring blast of flame, which is directed against the painted surface
gh to soften the paint which is at once removed with a scraper
The paint is not actually burned, but only softened by the flame;
er, be removed as well as softened by this method. Houses cov-
chy wood, like southern pine, sometimes require this treatment,
ainting is found to be more lasting. In many cases it is sufficient
scrub the surface with a stiff steel-wire brush. Interior surfaces
d (if the removal of the old paint and varnish is necessary) with
er; this is a mixture of solvent liquids, which penetrate the old
sh and soften it, when it may be removed with scrapers or brushes.
anger of fire with this method than with the burning-off method,
r and costs more. It must not be forgotten that varnish-remover
highly inflammable and must not be used in a room where there is

always taken in painting wood, to secure a clean surfa
be omitted with steel. Mud and dirt must first be rem
it must be examined for rust, and any rust-spots mu
Loose scale may be removed with wire brushes, but t
rust must be removed with steel scrapers, or with hamn
No doubt the best way to clean steel is to use the sand-b
for much architectural work. In any case much care
clean surface. On wood the priming coat sinks into the
bond between it and the succeeding coats; but on m
sible and it is a case of simple adhesion, which deman
cient results. The paint for structural metal should
to as great a degree as possible it should be water-pro
should never be applied, and three are better. Paint is
angles, and also on bolt and rivet-heads; it is therefor
first full coat, to apply a partial or striping coat, cove
and the surface for at least 1 in back from the edge
heads and rivet-heads. After this striping coat has b
coat is applied, and it may then be assumed that the v
two full coats. At least a week should elapse betweer
steelwork, all cavities which may be filled with rain
properly drained; and during erection all small cavi
cement, and all contact-surfaces thickly painted.

Kinds of Paint for Structural Steel. RED LEAD is
anything else as a paint for structural steel. It is
usually made from litharge (PbO), and frequently cont
the latter. If it contains much litharge, it rapidly t
oil and finally hardens; this makes it a paint difficult t
material from which it is made is reduced to a sufficient
oxidized, an almost completely oxidized red lead is pr
worked as white lead, and better in every respect.
government of the United States have for years calle
than 94% of oxide of lead (Pb_3O_4), and the Navy Depa

lent results. Graphite is sometimes mixed with lampblack, prob-
ntage. Boneblack is also an important ingredient of CARBON
rly oxide of iron in linseed-oil was used more than all other paints
but while many engineers still like it, its use has very greatly di-
HALTUM has been used and is still used, as a varnish either alone
on, and some of these asphaltic preparations are fairly satis-
act is, that a really competent paint-manufacturer can make a
paint out of any of these, and if the paint is carefully applied
be satisfactory. There are great differences in painters. In re-
ace of structural steel covered by a gallon of paint, there is a
of opinion among experts. Some say from 300 to 400 sq ft,
1200 sq ft. The truth is that any paint may be brushed out
gly thin film by a skilled workman, while ordinary usage results
twice as thick. The general opinion is that it is not wise to esti-
400 sq ft to the gallon for one coat. Varnish-paints cover less
but if well made they are very durable.

Cement and Concrete. Cement and concrete-work are diffi-
ecause they are strongly alkaline and even caustic when new.
aterials should be allowed to stand a year or two if possible be-
l; then it may be painted with any ordinary paint. A practice
ighly recommended is to wash the surface, repeatedly if possible,
lution of zinc sulphate, the sulphuric acid uniting with the free
c being left in the pores as an oxide or hydrate. Some prepara-
rpose are on the market; and while some are probably good,
distrusted. The best way is to allow the surface to age, if this
.

WINDOW-GLASS AND GLAZING *

he glazing of windows originally belonged to the painter's trade,
is broken, it is still customary to go to a painter to have it re-
tom has so changed in some parts of the country, that when new
be glazed, the work is sometimes done at the mill or factory
are made, sometimes by the local glass-jobber in the town where
being erected, and again, in other localities, the glazing of new
done by the painter. COMMON WINDOW-GLASS is usually set
secured with triangular pieces of zinc called GLAZIERS' POINTS,
wood over the glass and covered with putty. In the best work, a
ty is first put in the rebate of the sash and the glass is then placed
d down to a solid bearing. This is called BACK-PUTTYING. The
driven about 8 or 10 in apart and the putty applied over the glass
to fill the rebate. Outside windows should always be glazed on
he sash. Common window-glass has a slight bend in it, the re-
nal cylindrical shape; it should be glazed, therefore, with the
as this reduces to a minimum the effects of the waviness when
it either from the outside or inside. Plate glass, in window-
-lights, should be back-puttied and secured by wooden beads.

. It was formerly a common practice for architects to name
ions a certain sum of money to be allowed by the carpenter for
and to be expended under the direction of the architect. Where

om article on Window-Glass and Glazing by Professor Thomas Nolan
truction and Superintendence, Part II, Carpenters' Work, by F. E.

location in the building.

Sheet Glass. General Description. Common win
known as SHEET GLASS or CYLINDER GLASS. "It is mad
ping a tube with an enlarged end in the molten glass or
lb are gathered up. Then it is blown out slightly by th
blowing-tube and still further blown and manipulated,
15 in in diameter and 60 in long is formed. This cyli
trimmed off and is then cut longitudinally and gradual
placed on a large flat stone supported by a carriage, wh
softens sufficiently to open out flat; the carriage is then
ing-chamber and the sheet taken off." About the year
by machinery, utilizing compressed air, was perfected, ar
gradual decrease in its cost. The cylinder blown by com
and flattened out in just the same manner and by the
mouth-blown cylinder.

Grades and Qualities of Sheet Glass. Sheet glass is gr
or SINGLE-THICK, and each thickness is further divided int
SECOND, or THIRD, according to its relative freedom from d
according to the strength and quality. It should be rem
is always wavy, the result of the flattening of the cylind
by designating sheet glass, CRYSTAL-SHEET GLASS, SEL
SHEET GLASS FREE FROM WAVES AND IMPERFECTIONS, a sh
and blemishes can be obtained. The terms and names d
of this glass, which still remains sheet glass, characterized
in the method by which it is manufactured. To obtain
waviness, plate glass, ⅛ in thick, sometimes known as c
glass 3/16 in thick, must be specified. Since the improv
ture of window-glass in this country, scarcely any sheet g
glazing purposes. A small amount of Belgian sheet g
country and used along the Atlantic seaboard for pict
prices of the American sheet glass, and its excellent q
forced imported sheet glass out of the market. All comm
regard to quality, is graded according to thickness, as st

iving the price for each size, in both thicknesses, and all qualities;
: prices a very large discount is allowed. Fluctuations in prices
y the discount, the list usually remaining unchanged for a number
present price-list (1913) has been in use since October 1, 1903.
o ascertain the price of a light of glass of a given size is to find it
list, from which the discount, quoted by the glass-dealer, must be
r the benefit of the Pacific Coast trade there is a Western Glass
ffers somewhat from the Eastern list. The list is for sheet glass,
lists being the same in the East and West. The price per square
apidly as the size of the pane increases, so that it is much cheaper
e window into eight or twelve lights than into two lights. Com-
cost of the building, however, the glass is a small item and in the
f buildings each sash is usually glazed with a single light of glass.
orkshops, etc., where there is usually a large amount of glass-
e of the lights is not of so much importance, while the saving by
hts is quite an item; hence twelve-light and even sixteen-light
nerally used in such buildings. The following table shows quite
tive cost (1913) per square foot of different-sized panes of Ameri-
rices given being about an average for the whole country.

**Cost (1913) of American Sheet Glass per Square Foot, Based
n a Discount of 90 and 20 Per Cent on the List of
October 1, 1903 ‡**

	Sizes of lights in inches						
	10×12	15×20	24×34	30×36	36×40	40×60	60×70
	Prices in cents per square foot						
th:							
…..	7.0	8.3	9.4	10.0	10.8	14.0	29.2
ity….	6.0	7.3	8.3	9.0	10.0	14.4	27.0
:							
…..	5.0	4.8	6.4	6.8	……	……	……
ity….	4.3	4.5	5.6	6.0	……	……	……

Glass, 26-Ounce. This glass is made by the cylinder-process,
hicker than the ordinary double-strength glass. It is probably
nade, next to plate glass, but owing to the method of its manu-
ssarily characterized by a wavy appearance. If good glass is
st-class residences, hotels, office-buildings, etc., polished plate
used. The latter invariably gives satisfaction, while sheet glass,
hat thickness, is usually disappointing in its appearance.

eet Glass. All sheet glass, when looked upon from the outside,
tery appearance, like the surface of a lake slightly agitated by

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ts of glass have been omitted as they can readily be obtained from the
ny city. Such lists are not of much service unless they are complete;
are too long to be inserted in a condensed handbook.

h discounts from the prices given, may be obtained from the W. P. Fuller
rancisco, Cal.

ble information in regard to Window-Glass and Glazing was furnished by
e of the *Hires-Turner Glass Company, Philadelphia, Pa.*

and an elegant appearance required on the outside. Th
of plate glass is entirely different from that of sheet glass
the metal, which is prepared with great care, is melted in
on a perfectly flat cast-iron table. "The width and t
determined by means of metal strips called GUNS, which
which a heavy, metal roller travels. The ends of the g
when the roller is at one extremity, it and the guns form
rectangular dish. The molten metal is poured on and
slowly, forcing the metal in front of it and rolling out t
then annealed and forms what is known as ROUGH PLATE
lights, skylights, floor-lights and the like. "For polish
is carefully examined for flaws, which are cut out, leavi
practicable. The plate is then fastened to a revolving t
of Paris, and two heavy shoes, shod with cast iron, are
table is then revolved and sand and water fed onto the s
also, going over all parts of the plate and grinding it
Emery-powder is then fed on, in successive degrees of f
made absolutely smooth and all grit removed. After
with very fine felt, are put on and liquid rouge is added f
one side is completed the other side is similarly treated
40% in weight by the operation."

Qualities of Polished Plate Glass. For glazing pu
quality of plate glass on the market. The best of this is
ing mirrors. At one time, plate glass was extensively i
ally improving methods of the American manufacture
cheapening of the process, have practically eliminated i
the market. The American plate glass is equal in eve
was imported. The usual thickness of polished plate g
but it can be made thinner than this; and when require
or car-windows, may be obtained in 3/16 or 1/4-in thicknes
from the same thickness of rough plate used for the ord
ground down thinner and, owing to the additional cost
the risk, is more expensive than glass of the ordinary th

ness on account of the extra expense of grinding it down. Plate
ick costs from 25 to 40% more than glass of the regular thickness.

lished Plate Glass. Plate glass is cut into stock sizes, varying by
s from 6 by 6 in up to 144 by 240 in, or 138 by 260 in.

ive Cost of Different Kinds of Window-Glass. The following
accurate an idea of the comparative cost of the different kinds and
lass used in this country for glazing as it is possible to give, the
sizes being the present (1914) net, average prices. The first column
ives the kinds of glass, and includes both the American plate and
sheet glass. The other columns of the table give the sizes of the
ts in inches.

omparative Cost of Different Kinds of Window-Glass

Kinds of glass	Sizes of lights in inches			
	24×32	30×36	36×40	48×60
American Plate Glass				
lity	$2.35	$3.38	$4.60	$9.80
t glass, 26-oz	1.00	1.54	2.34	6.66
American Sheet Glass				
ngth, first quality	0.54	0.83	1.25	3.55
ngth, second quality	0.47	0.73	1.13	3.30
gth, first quality	0.37	0.56	......	......
gth, second quality	0.32	0.50	......	......

een from this table that the relative difference in the cost of plate
ss decreases rapidly as the sizes of the lights increase. The prices
are based on the list of October 1, 1903, on a discount of 90% for
o and 20% for American sheet glass and 85% on AA double-
z crystal-sheet glass.

ss. This is described in Chapter XXIII, page 821.

olled Glass. This is a translucent or OBSCURED glass with a pat-
on one surface. As the molten metal is rolled out on the table,
t into the table, imprints itself into the soft glass. This kind of glass
tirely supplanted the ordinary ground glass because of its greater
There are several popular designs on the market, made by various
s. Some of the designs in common use are known as MOSS, MAZE,
ORENTINE, COBWEB, etc. This glass is usually made ⅛ in thick and
s from 24 to 42 in wide and from 8 to 10 ft long. MAZE, FLOREN-
WEB designs can be had either with or without the wire mesh in
mportant property of figured rolled glass is that of diffusing the
asses through it. (See, also, pages 1367 and 1368.)

rism-Plate Glass.* This is manufactured in different patterns and
purposes and includes (1) Imperial Prism-Plate Ornamental Glass
nt patterns, (2) Imperial Prism-Plate Glass and (3) Imperial Sky-
lass. The general description is as follows:

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red by the Pressed Prism Plate Glass Company, Chicago, Ill. See, also,
370.

is a plate glass, ground and polished on one side. The
collect dirt and retard the light and it is very easily c
upper sashes of windows and in transoms, store-fronts,
(3) Imperial Skylight Prism Glass. This is made in
with a ½-in back, and conforms to the requirements o
ance Underwriters. It is used for skylights, roofs ov
wells, etc. The possibility of leakage is lessened on
plates in which it may be obtained. These plates,
smaller sizes if required. It is particularly adapted f
of stores and for railway-stations, sheds, etc.

Prism Glass, for glazing windows, skylights and side
factured in a large number of forms in both prisms a
companies. The diffusing properties of several type
1367 to 1370 under the subject of Illumination. This
prisms which are glazed horizontally in the windows a
throw it back horizontally into the rooms, adding v
terior lighting. It is manufactured by several compa
from glass-jobbers in practically all the cities of the U
page 821.) Glass prisms for lighting are made of piec
mensions, about 4 in square, with a smooth outer surfa
vided into a series of prisms. They are, in many cas
the process of electroglazing, the edges of the prism-len
so to speak, by a narrow line of copper which gives
strength for use in large frames, and also an attractive
some to be superior to ordinary leaded work. These
in any desired size, but for very large surfaces two o
means of metal sash-bars, are generally used. (See, a
The commercial value of these prisms depends on th
causes what is known as REFRACTION. Prism-plates
sky, not necessarily from the sun, and refract or turn it
is to be lighted. With an ordinary window the lig
through the glass, strikes the floor at a point not very fa
As the color of the floor is usually dark, reflecting per

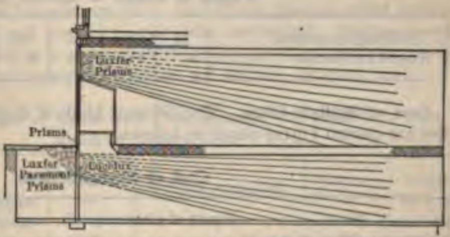
ight entering the window, but simply redistribute it, directing it
rtions of the room in which it is most needed. By thus changing
of light-rays a room with a low ceiling can be better lighted than
plate glass is used. To insure success in the lighting of interiors
prisms requires, however, a superior quality of glass, and careful
ulations and experiments, besides practical and attractive means
l methods of installation. These requirements have been met by
ompanies making these prisms and their products may be con-
g the relatively new building materials. They have been very
pplied to the lighting of dark rooms by daylight. The application
any particular building depends upon the surrounding conditions
ents, each case requiring some special treatment; but in a general
ous appliances used in the installations may be divided into four
ows:

l Plates, which are set directly in the sashes in place of the ordinary
They are commonly used for the transom-lights of store-windows
r sashes of double-hung windows. They may also fill the entire

es, which are vertical prism-plates set in independent frames and
dow-openings substantially flush with the face of the wall.

es, which are external prism-plates in independent frames, placed
openings and set at an angle with the vertical, a position similar to
linary awning.

ent-Prisms, which are set in iron frames in the pavements or side-
e of the ordinary bull's-eye lights. In connection with the pave-
when a
basement is
al plates of
below and
pavement-
ften used.
g, vertical
the light
pavement-
gain chang-
on, project
y into the
his feature
d in the
iven, reproduced through the courtesy of the Luxfer Prism



Refraction and Transmission of Light by Prisms

es may be made either stationary or adjustable and may be em-
ariety of ways, combining the useful with the ornamental. The
ical plates lend themselves to a highly decorative treatment. In
l and hanging vertical plates the prisms may be arranged to pro-
tal effects, and designs may be inwrought on the face of the prism-
espond with the designs worked into the surfaces of the building
style of the entire façade. The prism-plates weigh no more, and
n plate glass of the same size, while they are much stronger in
l-pressure; the action of hail and the impact of flying fragments.
smitting a very large amount of light, these prism-plates are not
the ordinary sense, and may thus be used as screens to hide un-
ws or to prevent persons looking either in or out of a window. At

thus saving a large amount of valuable floor-sp
rooms they effect a great saving in artificial light
no cost for maintenance. The extent to which th
architects, in both new and old buildings, shows
influence upon commercial architecture.

Glass for Skylights. General Description.
for skylights is either rough or ribbed skylight-glas
ing in the process of manufacturing glass with wire
being largely used for this purpose. The sizes
pitch of the skylight, small sizes being more desi
The weight of rough or ribbed glass, with or witho
as follows:

Weight of Rough or Ribbe

Thickness in inches........	⅛	3/16	¼
Weight in pounds..........	2	2½	3½

Cost of Skylight-Glass. The different kinds of
ties are quoted (1914) about as follows:

Cost of Skylight-Gla

Kinds of glass
Rough or ribbed skylight-glass, ⅛-in.............
Rough or ribbed skylight-glass, 3/16-in.............
Rough or ribbed skylight-glass, ¼-in.............
Rough or ribbed wire-glass, ¼-in.................
Maze, Cobweb, or Florentine wire-glass...........
Sheet prism glass.................................

precipitated in a film over the surface of the glass, thus giving it the reflecting. This film is afterward covered and protected by shellac, l paint. This modern method of manufacture has made it possible iirrors in considerably less time, and at a very much lower cost, than ifactured by the old-fashioned MERCURY-BACK process. There are claim that in spite of modern processes of manufacture, the old oduced the best results as far as durability is concerned. This is by the following statement inserted by Mr. Kidder in the preceding the Pocket-Book: "There are two kinds of mirrors on the market, time reliable mercury-back mirror, the other the nitrate of silver, or ter known to the trade as the patent-back mirror. The latter is now recent years, been most extensively sold as a substitute for the former. iufacture of mercury-back mirrors no chemicals are used, only two cury and tin-foil. The affinity of mercury for tin forms an amalgam to and not affected by the atmosphere. A mercury-back mirror is considered to be the only durable and permanent mirror. A nitrate-patent-back mirror is produced by the precipitation of a chemical nitrate of silver and other media on the surface of the glass, to which e coat of shellac varnish overlaid with one or more coats of paint. , irrespective of the quality of the glass from which it is made, will eriorate from the date of its manufacture to that of its final collapse, occur at any time from a few months, but certainly within a few

MEMORANDA ON ROOFING

.* The best shingles are those made from cypress, cedar, redwood, yellow pine and spruce, in the order mentioned. Redwood, while quite as durable as cypress, is less inflammable; sawed pine shingles to cedar, and spruce shingles are not suitable for good work.

er and Weight of Cedar and Pine Shingles Per Square of One Hundred Square Feet

ssumed vidth, in	Weather or gauge, in	Number of shingles per square †	Weight per square		Number of nails per square	Weight of nails per square, lb
			Cedar, lb	Pine, lb		
4	4	900	210	233	1 800	4.50
4	4½	800	200	222	1 600	4.00
4	5	720	192	213	1 440	3.60
4	5½	655	197	218	1 310	3.28
4	6	600	200	222	1 200	3.00
4	6½	554	203	226	1 108	2.77
4	7	515	206	229	1 030	2.58

Shingles. Cedar and redwood shingles as commonly sawed are 20 in ad cypress shingles usually from 20 to 24 in long, the longer ones allow-

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e complete information see Kidder's Building Construction and Superinten-II, Carpenters' Work, pages 321 to 325.

for waste, add from 6 to 10%, the greater allowance being for the shorter

than cedar shingles. With a rise to the roof of from
shingles, or any shingles 16 or 18 in in length, should
the weather; with a rise from 10 to 12 in, from 4½
and on steeper roofs they may be laid from 4½ to 5
be laid ½ in more to the weather. Some authoritie
posures for these lengths. Where the longer shingle
the weather may be increased up to 7 in for the 24-
shingles are commonly laid 5 in to the weather, and

Labor. An average shingler should lay 1 500 sh
work; on irregular roofs with dormers, 1 000 per 9

Nails. It requires about 5 lb of threepenny or 7
1 000 shingles.

Slate Roofs

Characteristics of Good Slate. A good slate sho
If the slate is too soft, however, the nail-holes will be
will become loose. If it is too brittle the slate will fl
squaring and holing and will be easily broken on the
give out a sharp metallic ring when struck with the k
under the slater's axe; should be easily HOLED with
should not be tender or friable at the edges." The
should have a bright metallic luster and be free from
faces. Very few of the Vermont slates, however,
ribbons. Most slates contain ribbons or seams w
approximately parallel directions. Slates containing
and should not be used in good work.

Color. The color of slates varies from dark blue,
gray and green. There are also a few quarries of r
slate does not appear to indicate the quality. All
black as is also that quarried in Virginia, while that
and Maryland is also black but borders on dark blue
firms as dark blue. Slate quarried in New York Sta

lates. The Monson, Me., slates and Brownville, Me., slates
ollows: No. 1. Every sheet to be full 3/16 in thick, both
d all corners full and square. No pieces to be winding or

ess may vary from 1/8 to 1/4 in, all corners square, one side gener-
side generally rough, no badly warped slates.
a., slates are graded:
A pure slate without any faults or blemishes.
As well made as No. 1 Clear, except that it contains one or
black band or streak across the slate), which, however, are high
te to be covered when laid, thus presenting a No. 1 roof.
This contains several RIBBONS, some of which cannot be
l.
A slate without RIBBONS, made from rough beds.
clear Bangor slate, not quite as smooth as No. 1 Clear, but
No. 2 Clear.
Slate. A smooth slate similar to No. 1 Clear, but bent at a
ft.

merly nail-holes in slates were punched on the job; now, how-
red and countersunk at the quarry, when so ordered. Archi-
ys specify that the slates are to be bored and countersunk, as
amages the slates.

es of slates range from 9 by 7 in to 24 by 14 in, there being some
ent sizes; the more common sizes, however, are the following:
best adapted for plain roofs are the large wide slates, such as
12 in, 20 by 12 in, or 24 by 14 in. Slates from 8 by 16 to 10
lar sizes, 9 by 18-in slates being probably used oftener than
size. The 11 by 22 and 12 by 24-in slates are used principally
buildings. The lower grades of slate are used largely on ware-
The larger sizes make fewer joints in the roof, require fewer
h the number of small pieces at hips and valleys. For roofs
sections the smaller sizes, such as 14 by 7 in or 16 by 8 in, look

tes vary in thickness from 1/8 to 3/8 in; 3/16 in is the usual thick-
sizes (see Grading of Slates in the preceding paragraphs). It
rtance for architects to specify the thickness of slates, either
or fully 1/4 in thick, to secure a strong and durable roof.

s are laid either on a board sheathing (rough, or tongued and
with tarred or water-proof paper or felt, or on roofing-laths
le and from 1 to 1 1/4 in thick, nailed to the rafters at distances
gauge of the slates. Each slate should lap the slate in the
ow, 3 in. The slates are fastened with two threepenny or four-
ear each upper corner. For slates 20 by 10 in or larger, four-
d be used. Copper, composition, tinned, or galvanized nails
Plain-iron nails are speedily weakened by rust, and they
the slates to be blown off. On iron roofs slates are often
small iron purlins spaced at suitable distances apart to receive
ed with wire or special forms of fasteners. THE GAUGE of a
on exposed to the weather, which should be one-half the re-
by subtracting 3 in from the length of the slate. Roofs to be
should have a rise of not less than 6 in to the foot for 20-in or
n for smaller sizes.

slates are laid.

Old English Method of Laying Slates.* This
volves the use of different shades of colored slates in
random widths beginning at the eaves, for example, wi
in thick, and using the different thicknesses from 1¼ t
in working upward on the roof. The use of this kin
creased in recent years and the method possesses v
ing out architects' ideas for varied artistic effects.
rough-cut edges in all thicknesses from 3/16 to 1½ in, i
shades carefully selected in such proportion as to pro
mony, when laid. As all of these colors and shades ar
effect is obtained at once and is permanent. These
usual sizes, but in the OLD ENGLISH STYLE, to be laid i
ferent lengths and in random widths. The Old Engli
ing-slates should be specified to secure the light-and-sh
utmost importance to specify the thickness desired, a
all sizes, while the cost varies according to thickness.
are desired, specifications should call for the number o
length and thickness beginning at the eaves courses,
are used in the largest sizes, sometimes 30 or even 36
upward on the roof with the shorter lengths and thinne
the smallest sizes and thinnest slates are used. To se
mum cost, specifications should call for Old English c
to be fully ¼ in thick with rough cut edges and gradu
ing from 24 by 16 to 12 by 6 in, with nail-holes dril
secure the best rough effect, specifications should call
than ¾ in thick, stating the thickness desired for the
courses desired in each length and thickness. Among
Old English style of roofing may be mentioned the b
versity for the Graduate College, where different shad
are used in thicknesses running from 1¼ in at the eav

Measurement. Slates are sold by the SQUARE, by
number of slates of any size to cover 100 sq ft of surf

ich the cost of laying is measured. "Eaves, hips, valleys, and cut- st walls or dormers are measured extra; 1 ft wide by their whole extra charge being made for waste material and the increased labor cutting and fitting. Openings less than 3 sq ft are not deducted, and around them are measured extra. Extra charges are also made for ires, and any change of color of the work and for steeples, towers, and ar surfaces." *

he cost of slates varies with the size, color and quality. The prices following table are about the average (1915) for blue-black slate, of e, loaded on the cars at the Pennsylvania quarry. The freight in s of 60-squares or over to Philadelphia from Bethlehem, Pa., is 60 are, from Pennsylvania to Omaha, Neb., $2.60 and from Vermont, ame. It will be seen that slates of the MEDIUM sizes cost the most, f the larger and smaller sizes the least. Special prices are quoted for . The larger sizes make the cheapest roofs. Red slates cost from more than black slates. The green slates are more expensive than ith the exception of the Maine and Peach Bottom varieties.

d Cost of Slates, and Pounds of Nails to 100 Square Feet of Roof

3-inch Lap

	Exposed when laid, in	Number to a square	Weights of galvanized nails,	lb	oz	Cost per square at quarry
	10½	98	4d	1	6	$4.50
	10½	115		1	10	4.50
	9½	126		1	12	4.75
	9½	138		1	15	4.75
	8½	155		2	0	5.25
	8½	170		2	6	5.25
	7½	160	3d	1	13	....
	7½	192		2	3	5.25
	7½	214		2	7	5.25
	6½	185		2	2	....
	6½	222		2	8	....
	6½	247		3	0	5.25
	6½	277		3	2	5.25
	5½	262		3	0	....
	5½	328		3	12	4.75
	5½	375		4	4	4.75
	4½	400		4	9	....
	4½	457		5	3	4.25
	4½	534		6	1	4.25

of blue-black-slate roofs, complete, varies from $9 to $16 per square, on the class of work and remoteness from the quarries. The addi- of laying slate in elastic cement varies from $1.75 to $2.50 per square. iced roofer will lay, on an average, 2½ squares of slate in 8 hours.

Slate roofing 3/16 in thick will weigh on the roof about 6½ lb per sq ft, thick, 8¾ lb, the smaller sizes weighing the most on account of the ictual weight of a square foot of slate ¼ in thick is 3.63 lb. A cubic

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* The Building Trades Pocket-book.

under former conditions have been covered with slate or n
a tile roof is as durable and satisfactory as one of No.
puted question. Mr. Kidder was of the opinion that, co
used, slates have given better satisfaction than tiles.
certainly more attractive than a slate roof, and it is ge
are many roofing-tiles on the market which if properly
durable as slates. There are so many patterns of roofin
sible here to enter into a description of them. Of the
which interlock are considered from a practical standp
satisfactory roof.

Laying Roofing-Tiles. Roofing-tiles have been lai
book tile or concrete base or on a sheathed surface over
been fastened to stripping over the sheathing or wooden o
of copper wires. When thus fastened by wires, the join
on the under side after they were laid, to prevent the
snow. Tiles of the older patterns were nailed to the she
method was superseded by the practice of fastening w
pierced lugs near the lower ends of the tiles. The best m
seems to be the one involving a solid continuous base for t
or not purlins are used. "Such purlins should be filled
book tiles or a concrete base and felt should be laid the
used, should be of a porous quality. Instead of regardin
a defective method, we have returned to it as the only p
ing tiles and have eliminated the stripping of sheathed ro
wires. Such methods would do in some portions of ce
winds and other climatic conditions are not severe, but
years' experience in the varied climatic conditions of the
found that the nailing of tiles with copper nails is the o
of application. We have also found that a roof should b
with a good asphaltum-felt to prevent wind-suction."
from 750 to 1 200 lb per square of 100 sq ft.

Specifications for Tile Roofin

ss otherwise selected by the architect. The tiles as specified
ard-burned, of red color, and in accordance with samples de-
:e of the architect.

of Roof. Before the roofer is sent for, the owner or general
nstruct the roofs in strict accordance with the plans, sheath the
all chimneys and walls above the roof-line completed, have all
ough the roofs, furnish all strips of required width used under
ll 1 by ⅞-in cant-strips used under the tiles at the eaves and
lding ready for the roofers' use. The metal-contractor is to
n place on the roof (gutters, whether box, hanging or secret
tend over the roof-sheathing and cant-strips, and run under
t least 8 in) and is to have in place, also, all valley-metal, the
to be not less than 24 in, with both edges turned up ¼ in
length of the valley. The valley-metal is to be fastened with
iled or punctured in any manner. The valley-metal is to be
r of felt running lengthwise the entire distance of the valley.
ctor is to have in readiness all flashing-metal used alongside
ormers, gables, skylights, towers and perpendicular walls, and
and chimneys, and is to place the same after the arrival of
l under his direction.

Felt. After the roofs have thus been prepared to receive the
tile-roofer is to cover the sheathing of the roofs with one thick-
ofing-felt weighing not less than 30 lb to the square, laying the
n lap and securing it in place by capped nails. The felt is to
th the eaves, lapped over all valley-metal about 4 in and laid
metal about 6 in.

Tiles. The roof having thus been prepared, the tile-layer is to
ith copper nails. The roofer is to see that the tiles are well
nd that they lie smoothly, and no attempt is to be made to
s. The tiles are to be laid so that the vertical lines are parallel
d at right-angles to the eaves. The tiles that verge along the
close against the hip-boards, and a water-tight joint made by
-tiles to the hip-boards with elastic cement. Each piece of
be nailed to the hip-board, and the hip-rolls are to be cemented
ch other. The interior spaces of hip-rolls and ridge-rolls are
th the pointing-material.

ıg-Tiles. The prices of tiles vary from $7 to $30 per square,
haracter of the surface-finish and to the pattern. The cost
g asphalt-felt, varies from $5 to $10 per square, according to
es used, the number of layers of felt and the character and
f. If roofing-tiles are laid on book tiles or on cement, 20%
the cost for laying on wooden sheathing. Fluctuating values
the item of copper nails, when these are used, one of im-

'iles. Roofing-tiles stamped from sheet steel, plain or galvan-
n sheet copper, in imitation of clay tiles, are made by several
d have been extensively used for factories and buildings of
ance. The first cost of these tiles, except those made of
ess than that of clay tiles and they do not require as heavy
n or galvanized-iron tiles, however, must be painted every
for a long period of years they probably cost as much as clay
n slate.

rolls. The pressure of these rolls leaves on the iron or st
which, to a great extent, determines the value of the pl
adjusted to leave a relatively large amount of coating c
coating, or a very scant coating. The heavier the coati
plate. Some makers employ a variation of this pater
plates are given an extra dip, by hand, in an open pot
FINISH. It is claimed that hand-dipped plates will last
made by the new process, although the latter process i
used and many good roofing-sheets are made by it.

Brands. The best roofing-plates always have the B
and as the manufacturers have a pecuniary interest in k
of these brands, the only way of being sure of a good tin
of tin that has a reputation for quality and durability.
brands are Taylor's Target-and-Arrow (formerly Old
Method, MF; Follansbee's Banfield Process; and M
plates are usually stamped with the weight of coating
by 20-in size.

Sizes of Sheets. The common sizes of tin plates are
of that measure. The sizes generally used are 14 by 20
larger size is the more economical to lay, and hence roo
for flat roofs the 14 by 20-in size makes the better roof

Thicknesses of Sheets. Terne-plates are made in
which the iron body weighs about 50 lb per 100 sq ft, an
62½ lb per 100 sq ft. For roofing, the IC, or lighter w
because the seams do not contract and expand as mu
thicker plates are used. For spouts, valleys and gutt
should always be specified, and should preferably be us
are stiffer and less liable to be dented or punched. T
does not add to the durability of the plates, as this depe
coating.

Weights of Sheets. The standard weight of 14 by 20-
lb for 112 sheets, the number usually packed in one bo

se the larger number of seams stiffens the surface and helps to pre-
and rattling in stormy weather. For a flat-seam roof, the edges of
e turned ½ in, locked together and well soaked with solder. The
tened to the sheathing-boards by cleats spaced 8 in apart and locked
Two 1-in barbed and tinned-wire nails are used in each cleat. No
be driven through the sheets. The seams must be made with great
cient time taken to properly SWEAT the solder into the seams. Steep
uld be made with STANDING SEAMS and with 28 by 20-in sheets.
re first single-seamed or double-seamed and usually soldered to-
rably end to end, into long strips that reach from eaves to ridge.
eams are composed of two UPSTANDS, interlocked at the upper edge,
he sheathing-boards by cleats. The standing seams are usually not
simply locked together with the cleats folded in about 1 ft apart.
be driven into the cleats only. The use of acid in soldering the
roof should be carefully avoided as acid coming in contact with the
the cut edges and corners, where the sheets are folded and seamed
ses rusting. No other soldering-flux but good rosin should ever be

of Tin Roofs. A tin roof of good material, properly put on, and
painted, will last from forty to fifty years, or longer. All traces
n the roof should be removed as soon as the tin is laid and soldered,
of paint should be applied promptly; a second coat should follow
ter the first. One or more layers of felt or water-proof paper should
der the tin, to serve as a cushion, and also to deaden the noise pro-
n striking the tin. The durability of tin roofing, and especially of
alleys and flashings, is generally increased by painting the tin on
ore laying. An excellent paint for tin roofs is composed of 10 lb of
, 1 lb of red lead and 1 gal of pure linseed-oil.

ce of Tin Roofs. The tin roof should be given one coat of paint
d and an additional coat of paint at four-year or five-year intervals
nply sufficient to keep its upper surface in first-class condition as
uilding stands. With each painting the roof is fully restored to its
ition. Graphite and tar paints should be avoided on tin roofs.
wn, Venetian red, red oxide or red lead, only, should be used as
th pure linseed-oil. Tinned gutters should be swept clear of accu-
leaves, dirt, etc., and if water has a tendency to lie in the gutters
be painted yearly.

Sheets Required to a Square. For FLAT-SEAM ROOFING a sheet of
in, with ½-in edges, measures, when edged or folded, 13 by 19 in,
but its covering capacity when joined to other sheets on the roof is
18½ in, or 231.25 sq in. The number of sheets to a square, there-
4 400 divided by 231.25, or 63, and an area of 1 000 sq ft requires 625
ox of 112 14 by 20-in sheets will cover, approximately, 180 sq ft.
20 in, when edged or folded, have a covering capacity of 490.25 sq in,
over 1 000 sq ft (10 squares) requires 294 sheets. For STANDING-
G the locks require 2¾ in off the width and 1⅛ in off the length of
A 28 by 20-in sheet, with the seams on the long edges, will cover 463
over 1 000 sq ft requires 312 sheets.

of Tin Roofing varies from $8 to $12 per square, according to the
tin, the locality and nature of the work and the scale of wages.
m roofs cost about 50 cts a square less than flat-seam roofs. The
by 20-in sheets are used, is about *25% more than for 28* by 20-in

sheathing-boards of well-seasoned dry lumber ar
widths are preferable, and the boards should be free fr
ness. A new tin roof should never be laid over old
roofs. Sheathing-paper is not necessary where the
above. If steam, fumes, or gases are likely to reac
some good water-proof sheathing-paper, such as black
used. Tarred paper should never be used. No nail
the sheets.

Flat-Seam Tin Roofing. When the sheets are l
fastened to the sheathing-boards by cleats, using thr
long side and one on the short side. Two 1-in barbe
to each cleat. If the tin is put on in rolls the sheets
lengths in the shop, and the cross-seams locked tog
solder. They should be edged ½ in, and fastened to
8 in apart, and the cleats locked into the seam and fa
1-in barbed-wire nails to each cleat.

Standing-Seam Tin Roofing. The sheets should
lengths in the shop, and the cross-seams locked tog
solder. They should be applied to the roof the narr
cleats spaced 1 ft apart. One edge of the course is
angle, and the cleats are installed. The adjoining
turned up 1½ in, and these edges are locked, turned
to a rounded edge.

Valleys and Gutters. These should be lined wit
flat seams, the sheets being applied the narrow way.
good solder, bearing the manufacturer's name, is use
half tin and one-half lead, new metals, and that not
flux. The solder should be well sweated into all sea

Painting. All painting should be done by the
painted one coat on the under side before it is appli
surface of the tin roof should be carefully cleaned of al

rshoes when on the roof. Wherever the slope is steep enough the tin
iid with standing seams, which allow for expansion and contraction.

Sizes, Weights, Etc., of Roofing-Tin *

in is usually furnished in two sizes, sheets 14 by 20 in and 28 by 20 in,
sheets to the box. Target-and-Arrow tin is furnished in three thick-
hickness, approximately No. 30 gauge, U. S. Standard; IX thickness,
ely No. 28 gauge, U. S. Standard; 2X thickness, approximately No.
. S. Standard, etc. Weight per 100 sq ft laid on the roof, about 65 lb
ness.

Covering Capacity of Roofing-Tin

a Tin Roofing. The following table shows the quantity of 14 by 20-in
to cover a given number of square feet with flat-seam tin roofing. A
20 in with ½ in edges measures, when edged or folded, 13 by 19,
, but its covering capacity when joined to other sheets on the roof
by 18½ in, or 231.25 sq in. In the following table each fractional
eet is counted a full sheet.

are feet.	100	110	120	130	140	150	160	170	180	190	200
uired...	63	69	75	81	88	94	100	106	112	119	125
are feet.	210	220	230	240	250	260	270	280	290	300	310
uired...	131	137	144	150	156	162	169	175	181	187	193
are feet.	320	330	340	350	360	370	380	390	400	410	420
uired...	200	206	212	218	224	231	237	243	249	256	262
are feet.	430	440	450	460	470	480	490	500	510	520	530
uired...	268	274	281	287	293	299	305	312	318	324	330
are feet.	540	550	560	570	580	590	600	610	620	630	640
uired...	337	343	349	355	362	368	374	380	386	393	399
are feet.	650	660	670	680	690	700	710	720	730	740	750
uired...	405	411	418	424	430	436	442	448	455	461	467
are feet.	760	770	780	790	800	810	820	830	840	850	860
uired...	474	480	486	492	499	505	511	517	523	530	536
are feet.	870	880	890	900	910	920	930	940	950	960	970
uired...	542	548	554	561	567	573	579	586	592	598	604
are feet.	980	990	1000	...	...	...	...	...	...	...	...
uired...	610	617	625	...	...	...	...	...	...	...	...

box of 112 sheets 14 by 20 in laid in this way will cover 180 sq ft.

a Tin Roofing. The following table shows the number of 28 by 20-in
red to cover a given number of square feet with flat-seam tin roofing.
ms edged ½ in take 1½ in off the length and width of the sheet. The
pacity of each sheet is, therefore, 26½ by 18½ in, or 490.25 sq in. In
g table each fractional part of a sheet is counted a full sheet.

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wing tables of sizes, weights, covering capacities and costs are adapted from
ompiled for the use of sheet-metal workers by the N. & G. Taylor Company,
Pa.

No. of square feet.	760	770	780	790	800	810
Sheets required...	224	227	230	233	235	238
No. of square feet.	870	880	890	900	910	920
Sheets required...	256	259	262	265	268	271
No. of square feet.	980	990	1000	....	....	....
Sheets required...	288	291	294	....	....	....

A box of 112 sheets 28 by 20 in laid in this way w

Standing-Seam Tin Roofing. The following table sh
20-in sheets required to cover a given number of squar
roofing. The standing seams, edged 1¼ and 1½ in, t
and the flat cross-seams, edged ⅜ in, take 1⅛ in off the
covering capacity of each sheet is, therefore, 11¼ by 1
the following table each fractional part of a sheet is

No. of square feet.	100	110	120	130	140	150
Sheets required...	68	75	82	89	95	102
No. of square feet.	210	220	230	240	250	260
Sheets required...	143	150	156	163	170	177
No. of square feet.	320	330	340	350	360	370
Sheets required...	218	224	231	238	245	251
No. of square feet.	430	440	450	460	470	480
Sheets required...	292	299	306	312	319	326
No. of square feet.	540	550	560	570	580	590
Sheets required...	367	374	379	387	393	401
No. of square feet.	650	660	670	680	690	700
Sheets required...	441	447	455	462	468	475
No. of square feet.	760	770	780	790	800	810

Seam Tin Roofing. The following table shows the number of 28 by required to cover a given number of square feet with standing-seam he standing seams take 2¾ in off the width, and the flat cross-seams, take 1⅛ in off the length of the sheet. The covering capacity of s, therefore, 26⅞ by 17¼ in, or 463.59 sq in. In the following table nal part of a sheet is counted a full sheet.

are feet.	100	110	120	130	140	150	160	170	180	190	200
iired...	32	35	38	41	44	47	50	53	56	59	62
are feet.	210	220	230	240	250	260	270	280	290	300	310
iired...	65	68	71	74	77	80	84	87	90	94	97
are feet.	320	330	340	350	360	370	380	390	400	410	420
iired...	100	103	106	109	112	115	118	121	125	128	131
are feet.	430	440	450	460	470	480	490	500	510	520	530
iired...	134	137	141	144	147	150	153	156	159	162	165
are feet.	540	550	560	570	580	590	600	610	620	630	640
iired...	168	171	174	177	180	184	187	190	193	196	199
are feet.	650	660	670	680	690	700	710	720	730	740	750
iired...	202	205	208	211	214	218	221	224	227	230	233
are feet.	760	770	780	790	800	810	820	830	840	850	860
iired...	236	239	242	245	249	252	255	258	261	265	268
are feet.	870	880	890	900	910	920	930	940	950	960	970
iired...	271	274	277	280	283	286	289	292	296	299	302
are feet.	980	990	...	...	...	...	...	...	...	...	...
iired...	305	308	...	...	...	...	...	...	...	...	...

ox of 112 sheets 28 by 20 in laid in this way will cover 360 sq ft.

e Long or Short Way. Sheets 14 by 20 in can be laid either the t way. The best roof is made by laying the sheets the 14-in way; using the 28 by 20-in sheets, they should always be laid the 20-in , with the short dimension crosswise.

Cost of Roofing-Tin

Cost of Tin for Standing-Seam Roofing

Sheets 28 by 20 in. Price per box and per square foot

sts ...	$11.00	$11.50	$12.00	$12.50	$13.00	$13.50	$14.00	$14.50	$15.00	$15.50
am sts ...	0.0297	0.0310	0.0324	0.0337	0.0351	0.0364	0.0378	0.0391	0.0404	0.0418
sts ...	16.00	16.50	17.00	17.50	18.00	18.50	19.00	19.50	20.00	20.50
am sts ...	0.0432	0.0446	0.0459	0.0473	0.0486	0.0500	0.0513	0.0526	0.0540	0.0553
sts ...	21.00	21.50	22.00	22.50	23.00	23.50	24.00	24.50	25.00	....
am sts ...	0.0567	0.0580	0.0594	0.0607	0.0621	0.0634	0.0648	0.0661	0.0675	....

estimates do not include cost of laying. The cost, using 14 by 20-in sheets, o about 25% more than the cost, using 28 by 20-in sheets, owing to the er of seams. More tin, solder, cleats and work are therefore necessary.

Tin in Rolls, or Gutter-Strips

Number of sheets required per linear foot for 20 and

Feet	Widths		Feet	Widths		Feet	Widths	
	20	28		20	28		20	28
1	1	1	35	16	23	69	31	44
2	1	2	36	16	23	70	32	45
3	2	2	37	17	24	71	32	45
4	2	3	38	17	24	72	32	46
5	3	4	39	18	25	73	33	47
6	3	4	40	18	26	74	33	47
7	4	5	41	19	27	75	34	48
8	4	5	42	19	27	76	34	48
9	4	6	43	20	28	77	35	49
10	5	7	44	20	28	78	35	50
11	5	7	45	20	29	79	36	50
12	6	8	46	21	29	80	36	51
13	6	9	47	21	30	81	36	52
14	7	9	48	22	31	82	37	52
15	7	10	49	22	31	83	37	53
16	8	11	50	23	32	84	38	54
17	8	11	51	23	33	85	38	54
18	8	12	52	24	33	86	39	55
19	9	12	53	24	34	87	39	55
20	9	13	54	24	34	88	40	56
21	10	14	55	25	35	89	40	57
22	10	14	56	25	36	90	40	57
23	11	15	57	26	36	91	41	58
24	11	16	58	26	37	92	41	59
25	12	16	59	27	38	93	42	59
26	12	17	60	27	38	94	42	60
27	12	18	61	28	39	95	43	61
28	13	18	62	28	40	96	43	62
29	13	19	63	23	40	97	44	62
30	14	19	64	29	41	98	44	63
31	14	20	65	29	41	99	44	64
32	15	21	66	30	42	100	45	64
33	15	21	67	30	43	....	....	....
34	16	22	68	31	43	....	....	....

Cost of Tin in Rolls or Gutter-Strips

Labor, solder, paint, rosin and other materials not

A box of 112 sheets in 28-in roll will cover 175
A box of 112 sheets in 20-in roll will cover 248
A box of 112 sheets in 14-in roll will cover 350
A box of 112 sheets in 10-in roll will cover 496

Cost per box (28 by 20 in)..........	$10.00	$11.00	$12.00
Cost per linear foot, 28 in wide.....	0.05714	0.06285	0.06856
Cost per linear foot, 20 in wide.....	0.04032	0.04435	0.04838
Cost per box (28 by 20 in)	$16.00	$17.00	$18.00
Cost per linear foot, 28 in wide.....	0.09149	0.09711	0.10282
Cost per linear foot, 20 in wide.....	0.06450	0.06853	0.07256

ls. For the convenience of roofers and for rush-orders, Target-and-
ut up in rolls 14, 20 and 28 in wide. Each roll contains 108 sq ft
ft, 28 by 20-in sheets laid 20 in wide). The tin is painted on one
as wanted, with good metallic brown paint. Seams are carefully
and, good 100 to 100 solder and rosin being used as a flux.

Slag or Gravel Roofing

ary Gravel Roofing[*] over boards is formed by first covering the
roof with dry felt (paper) and over this laying three, four, or five
d or asphaltic felt lapping each other like shingles, so that only
of each layer are exposed. In laying roofs over concrete the dry
a mopping of pitch is placed directly on the concrete and the first
lt embedded in it.

ainst walls, chimneys, curbs of skylights, etc., is done by turning
n against the walls. Over this is laid an 8-in strip with half its
oof. The upper edge of the strip and of the several layers of felt is
o the walls by nailing wooden strips or laths over the felt and into
etal flashings to protect the felt are better than the wooden strips
used when possible. At the eaves and on all exposed edges,
tops should be used.

ethod of Slag or Gravel Roofing is to lay two plies of tarred felt,
ther 17 in, and then spreading a coat of pitch over the entire roof.
three more layers of felt are laid and then coated with pitch, into
hed slag or screened gravel is embedded.

ons for Pitch-Slag or Gravel Roofing. The following specifi-
describe the latter method more in detail and also the materials
used to secure a first-class job. These roofs are most efficient and
nparatively flat inclines. The usual built-up roof consists of suc-
of saturated felt cemented together and surfaced with coal-tar
t, into which is embedded the gravel or slag. Tile is also used as a
erial. The saturants used in the felt are generally coal-tar or
unds.

ication for Pitch-Slag or Pitch-Gravel Roofing Over Wooden Sheathing

ation should not be used when the roof-incline exceeds 3 in to 1 ft.
ckness of sheathing-paper or unsaturated felt weighing not less
oo sq ft, lapping the sheets at least 1 in.
tire surface lay two plies[‡] of tarred felt, lapping each sheet 17 in
ding one, and nail as often as is necessary to hold them in place
ining felt is laid.
tire surface uniformly with pitch.

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ations for ordinary gravel roofing, including flashing, see page 767 in
uction and Superintendence, Part II, Carpenters' Work, by F. E. Kidder.
and adapted from specifications published by the Barrett Manufacturing
nown, in their full form, as "The Barrett Specifications." They can be
ne manufacturers.
ern States the number of "plies" is construed to mean the total number
ng dry as well as saturated felt, and the terms 3-ply, 5-ply, etc., are here-
hat basis. In the Eastern States, 3-ply, 5-ply, etc., usually refers to the
of saturated felt. The total number of layers should always be specified
oubt as to the exact meaning of the term as used in the specifications.

Over the entire surface lay three plies of tarred felt, lapping e
over the preceding one and mopping with pitch the full 22 in or
that in no place felt touches felt. Do such nailing as is necessary
are covered by not less than two plies of felt.

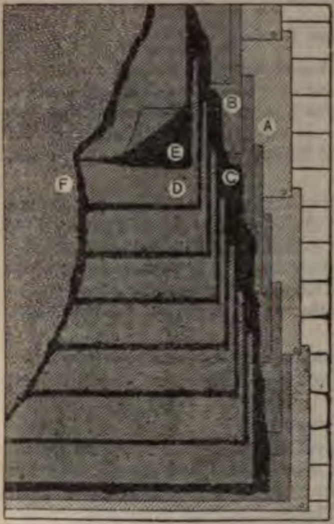

Diagram of Gravel or Slag Roofing on Wooden Sheathing

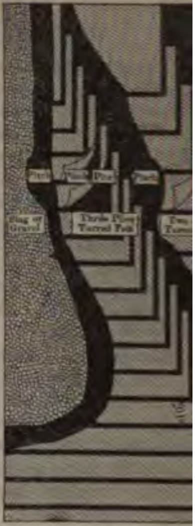

Diagram of Gravel or
Concrete I

Spread over the entire surface a uniform coating of pitch, i
hot, embed not less than 400 lb of gravel or 300 lb of slag to each
grains of the gravel or slag are to be from ¼ to ⅝ in in size, and d
dirt.

The roof may be inspected before the gravel or slag is applied,
not less than 3 ft long at right-angles to the direction in which
All felt and pitch is to bear the manufacturer's label.

(2) Specification for Pitch-Slag or Pitch-Gravel for Roofing

This specification should not be used when the roof-incline exc
foot. When the incline exceeds 1 in to 1 ft the concrete must per
nailing-strips must be provided.

Coat the concrete uniformly with hot pitch.

Over the entire surface lay two plies of tarred felt, lapping e
over the preceding one, mopping with pitch the full 17 in on eac
in no place felt touches felt.

Coat the entire surface uniformly with pitch.

Over the entire surface lay three plies of tarred felt, lapping e
over the preceding one and mopping with pitch the full 22 in or
that in no place felt touches felt.

Spread over the entire surface a uniform coating of pitch,

less than 400 lb of gravel or 300 lb of slag to each 100 sq ft. The
vel or slag are to be from ¼ to ⅝ in in size, and dry and free from
[illegible]
be inspected before the gravel or slag is applied, by cutting a slit
ft long at right-angles to the direction in which the felt is laid.
ch is to bear the manufacturer's label.

ag and Gravel Roofing. The difference between slag and gravel
or the former crushed slag is used instead of gravel. The greater
lies of tarred felt, the greater the amount of pitch that it is prac-
d it is the pitch that gives life to the roof. As there are several
s and qualities of tarred felt, a specification should state either
eight per 100 sq ft, single thickness (the most practical weight is
), or some known quality, such as Barrett's "Specification Tarred
ighing less than 12 lb per 100 sq ft is not economical even on the
To comply with the Barrett specification the materials neces-
o sq ft of completed roof are approximately as follows:

Over boards	Material	Over concrete
108 sq ft	Sheathing-paper	None
80 to 85 lb	Specification tarred felt	80 to 85 lb
120 to 160 lb	Specification-pitch	180 to 225 lb
400 lb	Gravel	400 lb
300 lb	Slag	300 lb

felt, the average weight is practically 15 lb per 100 sq ft, single
about 10% additional is required for laps. In estimating pitch
ditions and expertness of the workmen will affect the amount
e moppings and for a proper embedding of the gravel or slag. As
al qualities of pitch, a specification should either specify it by
"Specification-Pitch" or "Straight-Run Coal-Tar Pitch," or in
alt-pitch, the brand or origin should be plainly defined. The
-layer of sheathing-paper next to board-sheathing is mainly for
reventing any pitch which might penetrate the felt from cement-
to the sheathing. It is also of value in preventing the drying out
below through open joints. Where a less expensive roof is de-
or three plies of saturated felt may be used. With the four plies
used from 90 to 100 lb of pitch per 100 sq ft of completed roof;
ree plies from 70 to 80 lb of pitch.

f Slag or Gravel Roofs. These roofs, mentioned in the pre-
h, will last from five to ten years, or even longer, depending upon
e materials used and the care with which they have been applied.
strictly as provided for in the standard specifications will last
more, and if a tile surface is used, instead of gravel or slag, the
as long as the structure itself.

o Fire, Acid-Fumes, Etc. The fire-resisting properties of the
of are due principally to the incombustible material on the sur-
ned that the gravel or slag tends to prevent the successive layers
from burning and the whole mass has a blanketing influence on
within the building. *Some carefully conducted tests seem to*
vel roofing protects a wooden roof better than tin. The general
n gravel roofing is to soften the pitch or asphalt in the roofing.

to burn out the inflammable oil in them and to cause the re
a porous, incombustible coke. This type of roofing is not
gases or acid-fumes, and is used extensively on railroad-
structures where the conditions are particularly sever
should not be added to the pitch to soften it.

Guarantee. Roofers generally give a five-year guara

Cost of Pitch-Slag or Gravel Roofing. The cost
varies greatly, depending on the location, size and quali
tremes being approximately $2.50 and $3.50 per square
$4.50 per square for four-ply, and $4.50 and $7.00 per squ

Asphalt-Gravel Roofing

Asphalt-Gravel or Asphalt-Slag Roofing differs from
pally in the substitution of asphalt or asphaltic cement fo
saturating the felt as well as for mopping and surface-coat
the oils of asphalt do not evaporate as quickly as do those
ordinary temperatures and that therefore the flexibility a
and coatings are not as quickly destroyed. As a matter
not always last longer than some coal-tar roofs, but the
will last fully as long and possibly longer, depending u
materials and the workmanship. The asphalt used for r
cipally from the island of Trinidad.

Specifications for Asphalt Roofing.* The follow
prepared by the above-named company and are for W
Anchor-brand roofing. The manner of laying the felting
narily employed for coal-tar roofing.

(1) Specification for Asphalt-Gravel Roofing Over

Cover the roof with two thicknesses of Warren's
manila-paper side down, lapping each sheet 17 in over
securing with nails through tin discs about 2½ ft apart.

Over the entire surface of the Composite felt thus laid
of Warren's Anchor Brand roofing-cement, into which, w
ness of Anchor Brand felt, lapping each sheet at least 2 in
sticking these laps thoroughly with hot roofing-cement
through tin discs.

Over this first sheet of felt, mop again an even coating o
lay a second sheet of felt having the laps come in the mid
felt underneath, sticking the 2-in laps as before and secu
tin discs about 1½ in from the upper edge of the sheet.

Over the entire surface of the felt thus prepared, spread
cement, covering it immediately with a sufficient body
gravel or crushed slag.

If the roofing is applied in cold weather the gravel

cement, into which, while hot, lay two thicknesses of Warren's
oofing-felt, lapping each sheet 17 in over the sheet preceding.
the full width between the laps of the felt thus laid, with War-
and roofing-cement.
e exposed surface of the felt mop an even coating of said cement,
e hot, lay a third thickness of the felt, lapping each sheet at least
et preceding, and sticking these laps thoroughly with hot roofing-

d sheet of felt, mop an even coating of cement and over this lay a
el , having the laps come in the middle of the sheet underneath,
2-in laps as before.
e surface of the felt thus prepared, spread an even coating of the
g it immediately with a sufficient body of well-screened, dry
d slag.
is applied in cold weather, the gravel or slag must be heated.

halt-Gravel or Slag Roofing. Asphalt-gravel roofing costs a
pitch-gravel roofing of the same grade. (See Cost of Pitch-
Roofing, page 1512.)

.* Asphalt-gravel or asphalt-slag roofing should not be applied
re steep enough to make the material run in hot weather. The
f various roofings will guarantee the permanency of their roofings
imum slopes.

ofing. There is a large number of so-called PREPARED ROOFINGS
NGS, which are made by cementing together two, three, or more
ted felt or felt and burlap and then coating the combination either
tion of the same cementing material, or with hot pitch or asphalt
bedded sand or fine gravel. These roofings are commonly put
wide and are applied by lapping the strips 2 in with a coat of
rial between, and nailing every 2 or 3 in with tin-capped roofing-
ent quantity of cement, nails and tin caps is packed in the middle
particular advantage of these roofings is that no previous expe-
d for laying them and no kettles are required; for this reason they
used in the country, and on railroad-shops, factories, and mill-
cities there is no particular advantage in using them except for
o steep for coal-tar pitch, as they cost on the roof about the same
oofing. Many of these prepared roofings are as durable under or-
s as the light-weight gravel roofs. In Colorado, however, it has
they are badly damaged by severe hail-storms, probably owing
he protecting gravel. For roofs having a rise of 1 in or more to
roofings make economical and durable roofs, and for some build-
referred to other materials.

Corrugated Iron and Steel Sheets

Sheets of iron and steel are very extensively used for the roofing
ills, sheds, grain-elevators and warehouses. The best grades of
ts are now made of double-refined box-annealed iron or steel.†

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as been notified by the Warren Chemical & Manufacturing Company,
hen put on according to their directions, their Anchor Brand roofing has
used on relatively steep surfaces *where the slope was as high as* 9 in to the

hat "the life of a genuine PUDDLED-IRON *sheet* when exposed only to
ural elements is from five to eight times longer, and when exposed to

The corrugations are usually made lengthwise of the sheet, either b them through rolls or by pressing the plain sheets in a press made t desired corrugations. It is claimed that the latter method gives the fect and uniform corrugations. The weight and thickness of the m resented by the gauge-number of the black sheets from which the sheets are made. The standard gauge* for sheet iron and steel in th is that established by act of Congress, March 3, 1893.

Gauges. The following table gives the weights and thicknesses of ent gauges, from No. 7 to No. 30, for flat BLACK SHEETS. The gau from No. 7–0, ½ in thick, up to No. 40, 0.005469 in thick, but sheet commonly made thinner than No. 30, and above 3/16 in, the thicknes ally designated by fractions of an inch. Section 3 of the act of Co vides that in the practical use and application of this gauge, a v 2½% either way may be allowed.

United States Standard Gauge for Sheet Iron and Steel

Number of gauge	Thicknesses		Weights	
	Approximate thickness in fractions of an inch	Approximate thickness in decimal parts of an inch	Weight per square foot in ounces, avoirdupois	Wei squ in p avoi
7	3/16	0.1875	120	7.
8	11/64	0.171875	110	6
9	5/32	0.15625	100	6.
10	9/64	0.140625	90	5.
11	1/8	0.125	80	5.
12	7/64	0.109375	70	4.
13	3/32	0.09375	60	3.
14	5/64	0.078125	50	3
15	9/128	0.0703125	45	2.
16	1/16	0.0625	40	2.
17	9/160	0.05625	36	2.
18	1/20	0.05	32	2.
19	7/160	0.04375	28	1.
20	3/80	0.0375	24	1.
21	11/320	0.034375	22	1.
22	1/32	0.03125	20	1.
23	9/320	0.028125	18	1.
24	1/40	0.025	16	1.
25	7/320	0.021875	14	0.
26	3/160	0.01875	12	0.
27	11/640	0.0171875	11	0.
28	1/64	0.015625	10	0
29	9/640	0.0140625	9	0.
30	1/80	0.0125	8	0.

Galvanizing the Sheets adds approximately 2½ oz per sq ft to weights. The regular sizes of the corrugations are 2½, 1¼, ¾ a measured from center to center. Besides these sizes, 5-in, 3-in and

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sulphurous and other gases from ten to twenty times longer, than a sheet of st steel of the same gauge, or a light-gauge sheet made from pure puddled pig-iro it will wear longer than steel sheets of the heaviest gauges, or galvanized sh same gauge."

* For other gauges, see pages 402, 403, 1387, 1424 and 1426.

made by one or two corrugating companies. Corrugated sheets are
ock in 4-ft, 5-ft, 6-ft, 7-ft, 8-ft, 9-ft and 10-ft lengths. Sheets can
as long as 12 ft at a cost of 5% extra. The 8-ft length, however, is
nly used. The width of the sheets, as a rule, is 24 in between cen-
outer corrugations, so that the covering width is 24 in when one
is used for the side lap. This applies to all sizes of corrugations, all
r two mills make wider sheets. The 2-in, 2½-in and 3-in corrugated
ade in all gauges from No. 16 to No. 28, the 1¼-in corrugated sheets
to No. 28, the ⅝-in corrugated sheets from No. 24 to No. 28 and the
ated sheets in Nos. 26, 27 and 28 only. No. 28 gauge is the one com-
for all purposes. The sheets are generally painted with a red mineral
shipping and galvanized sheets, also, can be obtained if desired.
ed sheets are sold by the square (100 sq ft), measuring the actual
lengths of the corrugated sheets.

Corrugated-Steel Roofing *

ata. For covering roofs, either 3-in, 2½-in, or 2-in corrugations
ed, the 2½-in being the most common size. The thickness or gauge
n the distance between the supports on which the sheets are laid.
o 28 gauges should be laid on close sheathing, or strips not more
to 2 ft on centers. The maximum distances between supports for
should be as follows: †
4 gauge, from 2 to 2½ ft, center to center.
22 and 20 gauge, from 2 to 3 ft, center to center.
8 gauge, from 4 to 5 ft, center to center.
6 gauge, 5 to 6 ft, center to center.
pitch which should be given to roofs that are to be covered with
heets is 3 in to the foot, and for trussed roofs it is not desirable to

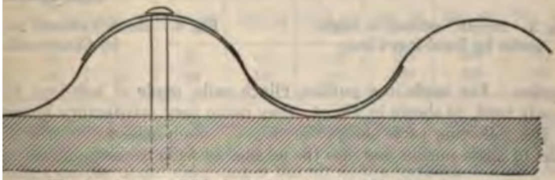

Fig. 1. Approved Method of Laying for Side Lap

an a one-fourth pitch (6 in to the foot). When laid on a roof, corru-
should have a lap at the lower end of from 3 to 6 in, according to
the roof. For a ⅓ pitch, a 3-in lap is used; for a ¼ pitch, a 4-in
r a ⅙ pitch, a 5-in lap. For the side lap it is recommended that
te sheet be laid upside down and lapped as shown in Fig. 1. By
, when water is blown through the first lap, it will stop and not pass
, but run down and out at the end of the sheet. A great deal of
ever, is laid as in Fig. 2. In applying to sheathing or wooden strips,
re secured by nailing through the tops of the corrugations, the nails
through every alternate corrugation at the ends, and about 8 in

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actical information regarding the *use of corrugated sheets* on mill-build-
ny details, is contained in Steel Mill Buildings and in the Structural En-
ook, by Milo S. Ketchum.
ength of corrugated sheets, see the books above mentioned.

apart at the sides. When applied to iron or steel purlins, th
extend over at least 1½ corrugations, and the sheets should b
every 8 in on the sides and at every alternate corrugation
Cincinnati Corrugating Company makes a patent edge-
makes a tight joint with a lap of only one corrugation. To f

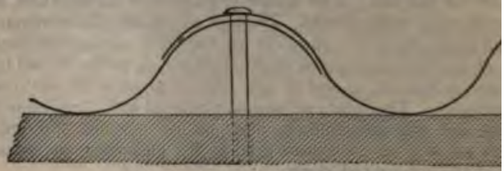


Fig. 2. Common Method of Laying for Side La

the purlins, which are usually steel angles, cleats of band-ir
may be passed around or under the purlins and riveted a
sheets, as shown in Fig. 3. By contracting or pressing thes
web, a tight, secure fastening results, which allows for contra
of the sheets. Cleats, however, are generally used only wit

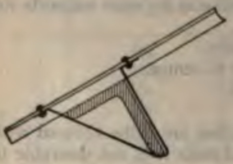


Fig. 3. Sheets Fastened to Angle-purlin by Band-iron Cleats

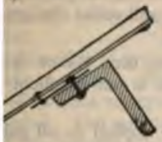
Fig. 4. Sheets Fast
by Clin

purlins. For angle-iron purlins, clinch-nails, made of soft-
monly used, as shown in Fig. 4; they make very satisfactory

The following table shows the sizes of clinch-nails to be
sizes of angle-purlins and also the number of nails to the pou

Purlin-angles	2×2 in	2½×3 in	3½×
Lengths of nails	4 in	5 in	6
Number of nails per pound	48	38	

corrugated sheets an allowance must be made for the laps. The
e gives the number of square feet necessary to cover one square of



. Approved Fastening
r Sheets at Eaves

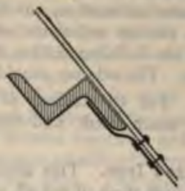


Fig. 6. Alternate Method of Fastening at Eaves

, using sheets 8 ft long. If shorter sheets are used, the allowance
ly increased.

Square Feet of Corrugated Sheets to Cover 100 Square Feet of Roof

....	1 in	2 in	3 in	4 in	5 in	6 in
	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft
rrugation.......	110	111	112	113	114	115
corrugations....	116	117	118	119	120	121
rrugations......	123	124	125	126	127	128

Weights in Pounds of 100 Square Feet of 2½-in Corrugated Sheets

No. 28	No. 27	No. 26	No. 24	No. 22	No. 20	No. 18	No. 16
69	77	84	111	138	165	220	275
86	93	99	127	154	182	236	291

nsation Lining. Wherever corrugated steel is laid on purlins
ing or paper underneath, if the building is heated, moisture will
ect on the under side, and if the air in the building is warm and
rable dripping will result. To prevent this dripping, it is neces-
the under side of the corrugated steel with paper or felt. This
y first stretching poultry-netting over the purlins, from eaves to
ng the strips together at the edges. Over this should be laid one
bestos paper and one or two layers of saturated felt. The cor-
nay then be fastened to the purlins in the usual way. The side
cured by stove-bolts, with 1 by $\frac{1}{8}$ by 4-in plate washers on the
support the lining.

Corrugated Siding

ither the 2½, 2, or 1¾-in corrugations are used. The 1¾-in size,
s the best appearance. For the laps, 1 in at the bottom and one
the sides are *sufficient.*

c., the sheets may be nailed to cross-pieces cut in between the
and spaced from 2 to 3 ft apart, the studs being from 3 to 4 ft

on centers. For elevators, either cross-corrugated sheets or than 32 in long should be used. The nails should be driven each alternate corrugation, 2 in above the lower end of the sh 1 in ABOVE the top end of the under sheet. This allows the in 32 in as the building settles, before the nail will strike the lower sheet. The side lap should not be nailed.

Ceilings. For the ceilings of stores, stables, etc., 3/16 or sheets are much used; and the construction is an excell purpose.

Galvanized Iron. This term is commonly applied to al metal, although most, if not all, of the galvanized sheets of th a steel base. The best quality of galvanized iron bears the tra or "Apollo Best Bloom." Galvanized sheets come in length in United States Gauge-Nos. 14, 16, 18, 20, 22, 24, 26, 27, widths of 24, 26, 28, 30 and 36 in for all gauges except No. 30, in widths of 24, 26 and 28 in. Sheets of No. 28 gauge are also 32 and 34 in. The widths commonly carried in stock are Most of the galvanized iron used for cornices and ornament gauge. No. 28 is sometimes used for gutters and conductors

Copper for Roofs

Method of Applying. This is usually in 2½ by 5-ft sh sq ft and weighing from 10 to 14 lb per sheet. It is laid on is fastened by copper cleats. No solder is employed, as it is horizontal joints, and the horizontal and sloping joints are ma lapping and bending the sheets. The horizontal joints are lo then tightly flattened down.

MEMORANDA ON TILING

Floor-Tiling and Wall-Tiling

Tile Floors are extensively used in the better class of b ticularly in those portions which are used by the public, on acc durability, sanitary qualities and decorative effects. As a ma tile floor is also cheaper in the long run than a wooden floor much wear. The materials used for tiling floors are tiles m grades of clay, marble, slate, glass and rubber. Of these p durable and sanitary are the vitreous clay tiles. For walls glazed tiles, marbles and glass are extensively used.

Floor-Tiles. The following include some of the principle

Tiles or Ceramic Roman Mosaic. This material is made of
in tesseral pieces representing the tesseræ of the Roman mosaics.
regular tiles ranging from ½ to ¾-in squares and also in hexagonal
¼ in to 1 in in size. A rounded LOZENGE TILE is also manu-
laid in tesseral paving. (See, also, Flooring of Mosaic, Terrazzo,
.)

l itself is of great hardness and well suited for work of a monu-
lic character. The even and regular texture of the tesseræ admits
of DAMASK DESIGNS which have become identified and associated
rial. The minuteness of the tesseræ admits of a great range in
the following of the architectural lines. The ceramic Roman
preferred to mosaic consisting of natural marbles, because of the
in colors and its greater durability. The vitreous-clay tiles are
attacks of any acids contained in the atmosphere, while marbles,
subject to rapid disintegration caused by the sulphuric acid con-
moke-laden atmosphere of our cities.

e Mosaics and Flint Tiles. These are the largest and heaviest
ured in this country. They are either plain or inlaid and are in
in ecclesiastic work on account of their relation to mediæval
The material is vitreous, annealed and tougher than it is brittle.
e for exterior polychrome work.

Tiles. These are large, heavy and thoroughly vitreous tiles used
ork. They are the only vitreous tiles of large size made in this
the tiles are large and generally of hexagonal shape, the joint-
uced to a minimum, and they are, therefore, especially adapted
operating-rooms and wards for contagious diseases.

Tiles, Wall-Tiles and Mantel-Tiles. The following include
ameled tiles:

Wall-Tiles. These are glazed tiles for wainscots. They have
ody and a surface covered with a clear glaze. The brilliancy of
its reflecting properties make the white wall-tiles especially de-
k passages.

Glazed or Enameled Tiles. These tiles are about the same as
quality; the GLAZE or ENAMEL, however, is stained with metallic
produces a brilliant decorative effect.

in, etc., Finished, Enameled Tiles. These are glazed tiles with
ND enamel-finish. The dull finish is produced either by sand-
devitrifying enamels. It is principally used for quaint decorative
tel-work.

Roman Mosaics. This is a type of enameled tiling which has
e possibilities. It has the same tesseral texture as the ceramic
is readily applied to wainscots and mantel-work.

Tiles. Clay tiles are set in Portland-cement mortar as a rule,
f this character should always be provided with a substantial
Ceramic mosaics are sometimes laid on a flexible base. With
on wooden floors can be provided with tile covering, and owing
y and lightness of the material, floors in elevators, boats and other
ctures can be safely tiled.

es, from 9 to 12 in square, have been extensively used for flooring,
count of their decorative effect. None of the marbles, however,
onsequently as durable as the vitreous and ceramic tiles, and

ment has set, the top surface is rubbed until it becomes
bleithic resembles mosaic or TERRAZZO, except that it is
instead of being put down on the floor in a plastic conditi
been made to TERRAZZO because of the cracks which co
to the slight settlements which are unavoidable in a n
Flooring of Mosaic, Terrazzo, etc., page 1521.) With t
the joints allow for any slight movement of the flo
causing visible cracks. By the process of manufactu
much harder than it is possible to make mosaic floors
condition, so that they have a much better wearing
material have been in use since 1895 and show little if
tiles are made of various colored marbles and in di
patterns, so that a great variety of effects may be pr
bases, stair-treads, and wainscotings, also, are made of

Cast-Glass Tiles, while quite resistant to a blow
broken, will break very easily when the surface is s
should, therefore, be very thick and small or protected

Novus Sanitary Glass* is a sanitary structural g
thicknesses from ½ in up to 2 in and in slabs of all width
in width and 180 in in length. It is made in various c
the following finishes: natural-fire finish, hone, sem
It can be worked and handled the same as marble, it is r
to accommodate fixtures, etc., and is very handsome i
pervious to discoloration and is non-crazing. These qu
desirable for floors, wainscoting, tables, shelves, etc., in
lutely sanitary condition combined with a handsome a

Interlocking Rubber Tiling

General Description. There is an interlocking r
cause of its being noiseless, non-slippery, and more com
inelastic substances, has met with great favor for f
counting-rooms, vestibules, elevators, stairs, cafés, libr

r, like a carpet, except that they are not fastened. Experience has they are very durable. Each tile is 2⅜ in square and ⅜ in thick; e required to the square foot. Rubber nosing for stairs is made to th the tiles.

Cost of Different Tiles

nate Cost. The following prices are approximately the cost, to the e factory. To these should be added the freight and the dealers' e cost of laying the tiles on a cement base, in addition to the cost should not exceed 25 cts per sq ft.

Floor-Tiles

Kinds of tiles	Factory price per sq ft
ncaustic tiles, unglazed	15 cts
es, white	22$\frac{1}{10}$ cts
e sizes	from 23 to 26 cts
es, or ceramic Roman mosaic	from 20 to 35 cts

Wall-Tiles and Mantel-Tiles

Kinds of tiles	Factory price per sq ft
d wall-tiles	23 cts
zed or enameled tiles	35 cts
iles, dull satin-finish	50 cts
, from 45 cts, upwards, laid	....
faience, plain colors	from $0.60 to $1

Flooring of Mosaic, Terrazzo,* etc.

of Mosaic Work is largely used. (See, also, Ceramic Tiles, or man Mosaic, page 1519, and Marbleithic Tiles, page 1520.) It is small pieces of stone, marble, pottery or glass, usually laid in some design or pattern. A bed of concrete is first laid and the small material used set in a floating of cement and made from ½ to 1 in n cubes of varicolored marble are used, pressed into the cement called ROMAN MOSAIC. A somewhat cheaper flooring is made by arble chips of irregular shape over the surface of the cement, m into it with plasterers' floats and rolling them with iron rollers. d TERRAZZO MOSAIC. The following is from the specifications for eld Museum, Chicago, Ill., D. H. Burnham, architects: "Filling zo shall be composed 1 part cement, 2 parts sand and 4 parts brick. ete filling commences to set spread a ¾-in wearing surface composed hips with only enough neat Portland cement to firmly unite the wel and roll, and after the mortar has set, rub the terrazzo to a surface and wash clean." "Terrazzo floors in the East cost from per sq ft, contractor's profit included." †

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e on Terrazzo Floors, by C. R. Marsh, in Journal of the Society of Construc- l Buildings, July, 1914.

om The New Estimator, by *William Arthur, 1914.*

ASPHALTUM

Bitumen, Asphaltum, Asphalt. "Bitumen is the r
group of mineral substances, composed of different hydr
diffused throughout the world in a variety of forms whic
tile liquids to thick semifluids and solids, sometimes in a
more frequently intermixed with or saturating differen
organic matter. To designate the condition under w
different names are employed; thus the liquid varieties
and PETROLEUM, the semifluid or viscous as MALTHA or
solid or compact as ASPHALTUM or ASPHALT." *

Asphaltum is found in extensive beds or lake-like dep
the most notable of these are the PITCH lakes on the isl
Bermundez, Venezuela. It is also found saturating t
stone formations in certain localities. Deposits of very
are found in Utah, Mexico, Cuba, and various parts
ELATERITE, GILSONITE and WURTZILITE are varieties of
tum.

Asphaltic Roofing-Materials are manufactured pri
asphalt. These deposits have also been the main sour
phaltum used in street-paving in the United States.

Rock-Asphalt. The term ROCK-ASPHALT is common
material obtained from the bituminous limestone depos
mont, in the valley of the Rhône, France, in the Va
Neuchâtel, Switzerland, and at Ragusa, on the island of
employed for paving purposes throughout Europe, and
much more durable pavement than can be made w
asphalt is prepared for shipment in two forms: (1) COMPR
which are used for paving in much the same way as stone
ASPHALT, which is put up in cakes of varying shape, gene
facturer's trade-mark.

Mastic-Asphalt. In the Eastern States MASTIC-ASPH
cellars, stores, breweries, malt-houses, hotel-kitchens,
servatories, public buildings, carriage-factories, sugar-ref
and for any place where a hard, smooth, clean, dry, fire
odorless and durable covering of a light color is required
or upper stories. It can be laid over cement concrete,
sheet without seams; also over cement concrete for roofs
For dwelling-house cellars, especially on moist or fille
especially adapted, being water-tight, non-absorbent, f
impervious to sewer-gases, and for sanitary purpose
asphalt is also valuable for DAMP-COURSES over founda
vaults and arches under ground.

er planks or boards, a layer of stout, dry, but not tarred, sheath-
first be put down and the asphalt laid on this. Asphalt floors
d be at least 1 in thick. Architects and owners desiring to em-
LT for any of the above purposes should be careful to secure the
-Travers, Seyssel, or Sicilian rock-asphalt, as there are
are of but little value.

ous Sandstones of California have been extensively used for
Western cities. They are prepared for use as paving-materials
powder. With this powder a considerable proportion of sand
rally mixed and the mixture heated until it becomes plastic; it
er the roadways and compressed by rolling.

MINERAL WOOL

ineral Wool. There are at least two kinds of mineral wool
untry. The more common quality is made by mixing certain
vith the molten slag from blast-furnaces and converting the
a fibrous state. The best slag for the purpose is that which is
The appearance of the finished product is much like that of
and fibrous, but in no other respect are the materials alike.
ade from slag appears in a variety of colors, principally white,
or gray, and occasionally quite dark. The color, however, is
dication of the quality, as all of the peculiar properties of the
ent in equal proportions in any of the shades. The other kind
is known as rock-wool, and is made from granite rock raised to
claimed that as it is absolutely free from sulphur, it is the only
anufactured. It has been approved by the United States War
t has the same general appearance as that made from slag, and

ineral Wool. Both of these materials consist of a mass of very
inelastic, vitreous fibers interlacing in every direction and
merable number of minute air-cells. Its great value in the insu-
tion of buildings lies in the number of air-cells which it contains,
on-conduction of heat, and its fire-resisting qualities. In wool
non slag, 92% of the volume consists of air held in minute cells,
grade the proportion of air reaches as high as 96%. This con-
t one of the best, if not the best, of the non-conductors of heat.
qualities it is very durable and contains nothing that can decay
y. Being itself incombustible it greatly retards the burning of
partitions if their inner spaces are filled with it.

eral Wool. The greatest value of this material is as an insula-
it is also a valuable non-conductor of sound. It is the general
r, that it can be considered only as a muffler of the sound-
seems to be no practical way in which it can be used so as to
the floor and ceiling. It would be crushed by laying floor-
As a muffler or filling between the beams, however, there is
g that is superior. In the end, then, it would seem that the
sulation from sound, without separate beams, would be obtained
flooring on some material like Cabot's Quilt or on a very thick
aces between the floor-cleats filled with mineral wool.

pplying Mineral Wool. Mineral wool, when used alone as
may be laid on boards cut in between the joists, or on top of
vhen that material is *used. The wool should be* at least 2 in

insulating material do no harm. When using mineral wo
packed in very closely, but not jammed so as to break
naturally very brittle. In partitions it is packed betwee
so as to completely fill the spaces, the wool being put i
reached a height of 2 or 3 ft. More laths are then put on
so on to the top. The wool should not be dropped from a
as the breaking up of the fibers destroys the insulating qu
In fact the tendency of mineral wool to settle and consol
too loosely packed, is the only drawback, except cost, to
The wool behind the lathing will not prevent the plaster

Cost of Mineral Wool. Mineral wool is sold by the
ing the quantity of wool required, 1 lb per sq ft of filling
allowed for ordinary wool and ¾ lb for selected wool. Th
wool is about $1.25, and of selected wool $2 per hundred

DATA ON STRUCTURAL ST

Estimating the Cost of Structural Steel fo

Structural steel for buildings is commonly made up
angles, Z bars and plates, which may be used as single be
into riveted girders, columns, or trusses. The Z bars ar
columns or other structural work in buildings. The cost
work is made up of the following items:

(1) Cost of the plain steel at the mill, plus freight and
(2) Extras for cutting, punching, fitting and assembling
or trusses.
(3) Cost of the fittings, such as connection-angles, guss
(4) Shop-painting.
(5) Cost of erection at the building.
(6) Painting after erection.

Base-Price of Steel. For orders of any considerable size

rrugated and checkered plates, from $1.75 to $1.90.
ates, structural, the base is $1.40.
ates, flange, the base is $1.50.
rrugated steel, painted, No. 22, $2.15.
rrugated steel, galvanized, No. 22, $3.00.
eel sheets, black, Nos. 10 and 11, $1.90.
eel sheets, galvanized, Nos. 10 and 11, $2.35.
teel sheets, black, No. 22, $2.10.
teel sheets, galvanized, No. 22, $2.95.
ar-iron, the base is $1.65.
ivets, $2.10.

ht-Rates (1913) in car-load lots are: Pittsburgh to Albany, N. Y., 16 cts; more, 14½ cts; to Boston, 18 cts; to Buffalo, N. Y., 11 cts; to Chicago, to Cincinnati, 15 cts; to Cleveland, 10 cts; to Columbus, O., 12 cts; ver, 85½ cts; to Louisville, 18 cts; to New York, 16 cts; to Norfolk, cts; to Philadelphia, 15 cts; to Richmond, Va., 20 cts; to Rochester, 11½ cts; to St. Louis, 23 cts; to Washington, D. C., 14½ cts.

Pacific coast points, it has been customary to allow a discount of about om the base, at Pittsburgh, on account of the high freight, and to meet an competition. On account of the expense of carrying beams in stock, ealers usually charge from ½ to 1 ct a pound, extra, on orders supplied ock.

of Extras to be Added to Prices of Plain Beams and Channels. kind of work whatever is done on the plain steel, or if the same is cut to with a less variation than ⅜ in, an extra price is charged, which is based following list, adopted in 1902. These charges are common to all shops rder is of any size:

Extras to be Added to Base-Price for Each 100 Pounds

or cutting to length with less variation than plus or minus ⅜ in	15 cts
lain punching one size of hole in web only	15 cts
lain punching one size of hole in one or both flanges	15 cts
lain punching one size of hole in either web and one flange or web and both flanges. The holes in the web and flanges must be of the same size	25 cts
lain punching each additional size of hole in either web or flanges, web and one flange, or web and both flanges	15 cts
lain punching one size of hole in flange and another size of hole in web of the same beam or channel	40 cts
unching and assembling into girders	35 cts
oping, ordinary beveling, including cutting to exact lengths, with or without punching; including the riveting or bolting of standard connection-angles	35 cts
or painting or oiling, one coat, with ordinary oil or paint	10 cts
ambering, beams and channels, and other shapes for ships or other purposes	25 cts
ending, or other unusual work	shop-rates
or fittings, whether loose or attached, such as angle-connections, bolts, and separators, tie-rods, etc.	$1.55

ods in all cases, where estimated upon in connection with beams or ls, are to be classified as fittings.

aking an estimate of the steelwork *from the framing-plans*, the weight

of all connection-angles, gusset-pl
off separately, and the cost figured
The standard connections are
and standard separators on page 6
In estimating the cost of rivete
bars and plates of which each col
an extra added to the price per po
This extra will be about as follo

Light channel-columns.....
Heavy channel-columns....
Plate girders, from 24 to 48 i
Box girders, from 24 to 48 in
Box girders, from 48 to 60 in

Cost of Erecting. For erecti
having masonry walls the cost o
there are bolted connections, and
erecting the steelwork of skelet
customary to allow $10 per ton.

Cost of Painting. The usual
but if done in accordance with th
this amount. For painting one
per ton.

Roof-Trusses. "In lots of at
in which the ends of the member
Trusses weighing 1 000 lb each, f
1 500 lb each, from $0.90 to $1.
from $0.75 to $0.85 per 100 lb;
from $0.60 to $0.75 per 100 lb.
per 100 lb more than riveted tru

Steel Mill-Buildings. The
buildings, including draughting,
from $15 to $25 per ton.†

Cost of Drafting. Details f
and valleys cost from $6 to $8 p
from $2 to $4 per ton. The cost
the character of the construction
architect's drawings. The aver
drawings are about as follows:

For entire skeleton constructi
foundations by the steel co
For the interior parts which ar
walls carry the floor-loads
For the interior parts which
outside walls carry the flo
For construction without colun
sonry walls, $0.85.
For buildings in which roof-tru
part of the construction, $

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* See Structural Engineers' Hand
† For much valuable data relatin
Buildings, by Milo S. Ketchum.

s in which roof-trusses on masonry walls comprise the greater the construction, $1.25.

ldings, average, $2.50.

turing or shop-buildings, with flat roofs, and one story in height,

ons, additions, remodeling, which require measurements before and shop-drawings can be made, $1.90.

e Estimates of the Weight of Steel in Buildings. Accord-yrrell,* the weight of steel in any proposed new building may be ted from the following data, which is a fair average for buildings n stories high, designed according to the Building Laws of the :

	Per sq ft of floor
t-houses and hotels, with outside frame	14 lb
t-houses, without outside frame	9 lb
dings, with outside frame	23 lb
dings, without outside frame	15 lb
es, with outside frame	28 lb
es, without outside frame	18 lb

s higher than eleven stories, the weight of floors will increase in ion to the number of stories, while the weight of columns will rapidly.

roximate weight of roof-trusses, see Chapter XXVII, pages 1050

rchant Steel. The cost of merchant iron and steel of all kinds ertain size of each particular shape, which is taken as the BASE, of all other sizes is figured at a certain extra rate above the g to a standard CARD OF MILL-EXTRAS. The BASE-PRICE may e changed without notice, but the extras remain constant, and n all localities. The following tables, on pages 1528 and 1529, ndard classification of extras on iron and steel bars.

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g Structural Steel, in Architects & Builders' Magazine, Jan., 1903.

Standard Classification * of Extras on Iron and Steel Bars

Adopted August, 1902

Rounds and squares †

Sizes	Extra per 100 lb	Sizes
3/4 to 3 in.	Base	3 1/16 to 3 1/2 in.
5/8 to 11/16 in.	$0.10	3 9/16 to 4 in.
1/2 to 9/16 in.	0.20	4 1/16 to 4 1/2 in.
7/16 in.	0.40	4 9/16 to 5 in.
3/8 in.	0.50	5 1/8 to 5 1/2 in.
5/16 in.	0.60	5 5/8 to 6 in.
1/4 and 9/32 in.	0.70	6 1/8 to 6 1/2 in.
7/32 in.	1.00	6 5/8 to 7 1/4 in.
3/16 in.	2.00	

Flat bars and heavy bands

Sizes
1 to 6 in × 3/8 to 1 in.
1 to 6 in × 1/4 and 5/16 in.
11/16 to 15/16 in × 3/8 to 3/4 in.
11/16 to 15/16 in × 1/4 and 5/16 in.
9/16 and 5/8 in × 3/8 to 1/2 in.
9/16 and 5/8 in × 1/4 and 5/16 in.
1/2 in × 3/8 and 7/16 in.
1/2 in × 1/4 and 5/16 in.
7/16 in × 3/8 in.
7/16 in × 1/4 and 5/16 in.
3/8 in × 1/4 and 5/16 in.
1 1/8 to 6 in × 1 1/16 to 1 3/16 in.
1 1/8 to 6 in × 1 1/4 to 1 1/2 in.
1 3/4 to 6 in × 1 5/8 to 2 3/4 in.
3 1/8 to 6 in × 3 to 4 in.

Light bars and bands

Sizes
1 1/2 to 6 in × Nos. 7, 8, 9 and 3/16 in.
1 1/2 to 6 in × Nos. 10, 11, 12 and 1/8 in.
1 to 1 7/16 in × Nos. 7, 8, 9 and 3/16 in.
1 to 1 7/16 in × Nos. 10, 11, 12 and 1/8 in.
13/16 to 15/16 in × Nos. 7, 8, 9 and 3/16 in.
13/16 to 15/16 in × Nos. 10, 11, 12 and 1/8 in.
11/16 and 3/4 in × Nos. 7, 8, 9 and 3/16 in.
11/16 and 3/4 in × Nos. 10, 11, 12 and 1/8 in.
9/16 and 5/8 in × Nos. 7, 8, 9 and 3/16 in.
9/16 and 5/8 in × Nos. 10, 11, 12 and 1/8 in.
1/2 in × Nos. 7, 8, 9 and 3/16 in.
1/2 in × Nos. 10, 11, 12 and 1/8 in.
7/16 in × Nos. 7, 8, 9 and 3/16 in.
7/16 in × Nos. 10, 11, 12 and 1/8 in.
3/8 in × Nos. 7, 8, 9 and 3/16 in.
3/8 in × Nos. 10, 11, 12 and 1/8 in.

* Intermediate sizes take the next higher extra. It is not customary to than one-half the "standard-card extras" for round and square bars.

† Squares up to 4 1/2 in only.

ndard Classification * of Angles, Channels and Tees

Angles

Sizes	Extra per 100 lb
nd heavier, but under 3 in	Base
in and heavier	$0.10
	0.20
	0.30
	2.00
	3.00
han ¼ in thick	0.50
nd larger, but smaller than 3 in, ⅛ in thick	0.10 per 100 lb over 3/16 in

Channels

Sizes	Extra per 100 lb
d heavier, but under 3 in	Base
n and heavier	$0.10
	0.20
in	0.30
	0.60
thicker	1.00
and wider, but under 3 in, ⅛ in thick	0.10 per 100 lb over 3/16 in

Tees

Sizes	Extra per 100 lb
d heavier, but under 3 in	Base
d heavier	$0.10
n and heavier	0.20*
hicker	0.50
hicker	0.60
hicker	2.00
arger, but smaller than 3 in, ⅛ in thick	0.10 per 100 lb over 3/16 in

* Intermediate sizes take the next higher extra.

car-load lots for any city may be obtained by adding the freight-page 1525 to the base prevailing at the mills.

MATING THE COST OF BUILDINGS *

ldings per Cubic Foot. The method of CUBIC-FOOT VALUES more than any other in estimating the cost of any proposed the plans and specifications are sufficiently complete for taking

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s indebted to E. S. Hand for valuable data relating to this subject. He to others whose names are mentioned in connection with particular e readers are referred especially to the Handbook of Cost Data for Engineers, by H. P. Gillette and to The New Building Estimator, by off the actual quantities. "Comparison of UNIT COSTS criterion by which to judge the economic merit of a struc method of doing work."[*] Two buildings in the same in the same style and for the same purpose, of the same same scale of wages and prices of materials, should cost the the same, per cubic foot, although one building may be som other and of different shape. It therefore follows that if CUBIC FOOT of different classes of buildings, in different loca imate quite closely the cost of any proposed building by contents in feet by the known cost per cubic foot of a sin built in that locality.

Size of Building Proportioned to Cost per Cubic a proposed building must be kept absolutely within a cer the building should be proportioned so that the CUBIC co ceed the quotient obtained by dividing the amount appropr COST PER CUBIC FOOT of similar buildings. Even then it the bids are opened, that they exceed the appropriation; a relatively small percentage of the total cost and the nec be made without altering the main features of the building

Methods of Computation. In estimating the cost by CONTENTS, it is of course necessary that the contents be basis, in both the proposed building and the one alrea contents are generally computed from the basement or cel age height of a flat roof, or, if there is a pitched roof, the attic is included, or that part which might be finished, me porches not being included. Vaults and areas under sid erally included as part of the basement. All measuremen of the walls and foundations. The estimated cost may o fees of the architect and other experts.

Other Methods of Estimating the Cost of Building ings, such as hospitals, theaters, schools, churches, barrac is sometimes estimated by the COST PER BED, SITTING, I are also based upon the COST PER SQUARE FOOT of GROU the FLOOR-SPACE, in certain types of buildings.

Data[†] on Cubic-Foot Values as a Basis for Preli of Building Costs

Notes on Modifying Conditions. Buildings of a give buildings and school-buildings, when similar in constructio under similar market-conditions as to cost of labor and m be nearly identical in CUBIC-FOOT COSTS. The buildings of differ widely in bulk, and this is always very considerable

ndicating that the results here are virtually identical; so nearly
AGE CUBIC-FOOT COST of a certain kind of building can be relied
estimate within from 3 to 5% of the actual cost of new work of
id under the same conditions. Other types of large structures,
ildings, hotels, churches and theaters, are less subject to stand-
se more variable in equipment and finish. This is true also of
and other small structures whose lesser bulk, moreover, renders
a close prediction as to their cost. These uncertainties do not,
t the rejection of the CUBIC-FOOT-COST METHOD for preliminary
ey do indicate that it is less closely approximate for some types
But the degree of uncertainty on even the most variable
nimized and should be reduced to perhaps 10% under a careful
mputation. Such system should cover a considerable number
ng account of all factors of material influence upon cost in each
ollow a consistently uniform method of determining cubic-foot

Which Influence Cost include the following.
market prices of labor and materials.
nstruction employed, depth and kind of foundations and exist-
special features such as towers or domes.
ternal facing and ornamentation; internal surfacing and deco-

t: (*a*) number and complexity of heating, lighting, ventilating,
, elevator and other systems; (*b*) extent to which apparatus
ment, such as laboratory-devices, opera-chairs, bank-counters,
rovided for direct use of occupants of building.
hitect and other experts.
Costs of structures of a given type will vary with the locality
of differing standards of practice and building laws, availability
ng materials, labor, etc.
s, developing in the experience of the architect.

of Determining Cubage may either simply recognize the
LUME of the building or, better, may employ a COEFFICIENT
y part whose cost varies materially from the average. The
ay be preferred as allowing a closer calculation of variations
mples. For instance, an unfinished cellar or other story or a
would cost less per cubic foot than the remainder of the build-
r or dome or finished basement containing, also, an expensive
t, would cost more. Foundations sometimes cost so much
figuring to their full depth as though the finished building were
that level.

osts. Subject to the foregoing considerations, the following
f buildings may be used as a STANDARD FOR AVERAGE CASES.
will probably be found too low for New York and some other
somewhat above those of several other sections of the United

steel and terra-cotta, stone and brick facings, complete equip-
r grade of interior finish:

Type of building	Cents per cubic foot
ngs.	32 to 35
ngs.	40 to 45
rs.	20 to 25

Construction: reinforced concrete; facing, common brick; equipment, typ usual in such structures; inside finish, the simplest.

Type of building	Cents per cubic foot
Factories[*]	14 to 16
Lofts[*]	15 to 18

Table [†] for Estimating the Approximate Cost of a New Building or the Value of an Existing Building

Based on prices for labor and materials in 1915. The cost of first-class fi proof buildings is greater in the Western and Southern States than in the Easter States, because of the distance from the great steel and material-centers.

Farm and Country Property[‡]

	Cost per cubic foot cents
Dwellings, frame; small box house, no cornice	
Dwellings, frame; shingle roof, small cornice, no sash weights, plain	6 to
Dwellings, brick; same class	5 to
Dwellings, frame; shingle roof, good cornice, sash weights, blinds (good house)	8 to
Dwellings, brick; same class (good house)	10 to
Barns, frame; shingle roof, not painted, plain finish	2 to
Barns, frame; shingle roof, painted, good foundation	3 to
Stores, frame; shingle roof, painted, plain finish	6 to
Stores, brick; shingle roof, painted, good cornice, well finished	8 to
Ordinary frame churches and schoolhouses; country	6 to
Brick churches and schoolhouses; country	9 to

If the roofs are slate or metal, add ¼ ct. per cu ft.

City and Village Property[‡]

Dwellings, frame; shingle roof, pine floors and finish, no bathroom or furnace, plain finish (good house)	7
Dwellings, brick; same class	9
Dwellings, frame; shingle roof, hard-wood floor in hall and parlor, bath, furnace, and fair plumbing	9
Dwellings, brick; same class	9
Dwellings, frame; shingle roof, hard wood in first story, good plumbing, furnace, artistic design, some interior ornamentation, well painted	11
Dwellings, brick; with good plumbing, bath, hot and cold water, pine finish, well painted, no hard-wood finish	1

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[*] If such subcontracts as plumbing, heating, lighting-fixtures, elevators, etc., included, of course these figures will be reduced. See Cost of Reinforced-Concret ings, page 1538.

[†] This table was originally compiled by Mr. F. E. Kidder from data collec various sources, and based on prices prevailing in 1902. The prices have bee to agree, as closely as possible, with those of 1915. All tabulations of this kind, costs per cubic foot are, at best, only approximate averages, and useful only a guides.

[‡] Lists similar to these were first compiled by James N. Brown of St. Lo and with the lower unit prices used in 1902 originally formed part of the furnished by insurance companies to their adjusters.

Miscellaneous Buildings		Cost per cubic foot, cents
laughter-houses		16 to 18
mplete, including patients' wards, administra- hapel, hospital, mortuary, laundry, workshops, essories		18 to 28
	$1300 to $1800	
te, or for barracks, but not t water		50 to 55
	$315 to $360	
rising swimming-baths, slip- ry, caretaker's quarters, ma- nplete		34 to 40
, including buildings, cellar- ne, machinery, coppers, liquor- ns, coolers, refrigerator, ice and all other requirements		16 to 23
		18 to 25
	$5.00 to $7.25	
	$45 to $60	
al		25 to 44
	$8.00 to $14.00	
	$73 to $135	
erally constructed		10 to 14
	25 to 34 cts	
te, with iron finishings and		16 to 18
	$2.46 to $3.14	
	$190 to $212	
with common fittings		12 to 15
	$1.85 to $2.25	
	$145 to $162	
for farm, wooden fittings		8½ to 11
	$1.62 to $1.68	
	$100 to $118	
or infantry		12 to 16
	$1.80 to $1.90	
f power-houses; buildings , exclusive of machinery and		16 to 19
in New York; comprising or- ork in front, elevators, fire- nd the whole well finished in hroughout		32 to 40
including administrative		23 to 34
	$1 750 to $2 600	
r small towns		19 to 25
	$1 200 to $1 750	
cluding all nursery-buildings.		19 to 25
	$2 000 to $2 600	
plete in every particular		35 to 46
		26 to 35
		23 to 27

Houses, first class; complete, in brickwork and good substantial finishings, large mansions with elaborate finish;	
Main building, 16-ft ceilings	
Per square foot	$6.25 to $7.25*
Additions, 11-ft ceilings	
Per square foot	$2.80 to $3.40
Houses, second class; large mansions of ordinary character;	
Main building, 14-ft ceilings	
Per square foot	$4.00 to $5.00
Additions	
Per square foot	$1.85 to $2.40
Houses, third class; country-houses;	
Height of ceiling, 11 ft	
Per square foot	$2.40 to $3.00
Houses, fourth class; speculative buildings;	
Ceilings, 10 ft	
Per square foot	$1.45 to $1.75
Houses, fifth class; tenements and cottages to rent;	
Ceilings, 9 ft	
Per square foot	$1.23 to $1.50
Hollow-tile houses, covered with cement or stucco, about	
Libraries; public, complete in every particular	
Municipal lodging-houses; for cities and large towns	
Per bed	$350 to $425
Museums, public; for large cities	
For towns	
Music-halls, complete; per head of accommodations, for large cities	$90 to $145
For small cities and towns	$45 to $80
Town-halls, complete; for large cities	
For small cities and towns	
Alternative prices; basement	
Superstructure	
Ornamental towers	
Theaters, complete; per head of accommodations, in large cities	$92 to $120
In small cities and towns	$56 to $90 or

Chimneys

Chimney-shafts, plain, as for factories, etc., complete, including foundations, iron cap, etc., height measured from surface of ground to top of cap; not exceeding 100 ft in height	$
Chimney-shafts from 100 to 180 ft in height	$
Chimney-shafts from 180 to 250 ft in height	$

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* The prices per square foot, in this and following paragraphs, are per squ floor-area, counting all of the floors above the basement. F. E. Kidder.

s of Actual Cost of Buildings per Cubic Foot

ate the subject further, a few examples of recently erected
. The lists were furnished through the courtesy of the
ildings. Such lists could be indefinitely extended, but those
ied sufficient to give some idea of the similarities and
based upon cubage. With the exception of reinforced-
it is probably true that for the past twenty or twenty-
1915) the cost of buildings has increased, with some varia-
increase, at the rate of about 1% per year.

s of the Actual Cost of Buildings per Cubic Foot

ese buildings were designed by Boring & Tilton

of building	Date	Height and type	Approximate cost	Cost per cubic foot, cents
me Insti- t, Md.	1900	Three stories and basement, fire-proof	$150 000	16
Library,	1908	One story and basement, non-fire-proof	85 000	17
r School,	1908	Two stories and basement, non-fire-proof	50 000	15
l, 87th St. ve., New	1907	Six stories and basement, fire-proof	275 000	29
's Friend West Sts.,	1909	Five stories and basement, fire-proof	200 000	32
Branch klyn, N.Y.	1906-1909	Six stories and basement, fire-proof	255 000	27
al, Tarry-	1910	Two stories and basement, non-fire-proof	65 000	26
ntingdon,	1909	Three stories and basement, fire-proof	90 000	20
Elizabeth,	1912	Three stories and basement, fire-proof	100 000	35
y, Spring-	1908	Two stories, mezzanine and basement, fire-proof	350 000	35
ry, Sioux	1912	Two stories and basement, non-fire-proof	75 000	21½
New York	1912	Twelve stories and basement, fire-proof	350 000	45
Immigrant land, New ain build-	1898	Two stories and basement, fire-proof	625 000	17½
	1898	Three stories and basement, fire-proof	150 000	28
's College,	1912	Three stories and basement, fire-proof	250 000	22

These buildings were designed by Palmer, Hornbostle & Jones

Name and location of building	Date	Notes	Cubic contents, cubic feet	Approximate cost	[illegible]
Oakland City Hall, Oakland, Cal.	1914	Based on all contracts except for lighting-fixtures	2 999 442	$1 400 000	[illegible]
Allegheny County Soldiers' Memorial, Pittsburgh, Pa.	1911	....	2 855 892	913 721	[illegible]
New York State Education Building, Albany, N. Y.	1912	Original contract completed, Dec. 1912	11 281 691	3 744 511	[illegible]

These buildings were designed by Robert D. Kohn

Name and location of building	Date	Height, character of construction and finish	[illegible]
Hermitage Hotel, New York City.	1907	Size of lot, 50 by 100 ft; height, 150 ft; 15 bedrooms and 11 baths on each floor; basement and sub-basement; power, electric light and refrigerating-plants; complete kitchen-equipment; brick and limestone exterior; cement floors.	[illegible]
Trades School Building, Manassas, Va.	1910	Main wing, 50 by 100 ft, two and one-half stories; shop-wing, 75 by 105 ft, one story, brick; shops, mill-constructed roof, cement floor; brick exterior throughout; heating-plant in extension; common wooden-floor construction, tin roof, class-rooms plastered; cheaply built.	[illegible]
Ethical Culture Meeting House.	1910	Height, 100 ft; basement, assembly-room; main floor, auditorium for 1 200 people; two stories above auditorium, Sunday school and offices; limestone exterior; fire-proof construction; oak finish.	[illegible]

Cost of Some Notable Buildings in New York City. Some of the prominent buildings in the Borough of Manhattan, City of New York, are cluded in the following table. For all these structures the permits were in 1911. By reason of its height the Woolworth Building may be the most notable of the list. It is not only the highest building in New City or the United States, but in the world. The cubic contents total 12 000 000 cu ft. Its foundations are carried to rock, which is about below the street-surface. The approximate weight of its steel frame is tons.

Some Notable Buildings in New York City[*]

Name of Building	Description	Heights, ft	Ground-areas, sq ft	Total floor-areas, sq ft	Esti-mated cost	Cubic contents,† cu ft	Costs per cubic foot,† cents
Hotel McAlpin	25-story hotel	307	31 100	690 000	$4 500 000	9 100 000	49
Grand Central Station	8-story railroad-station	148	118 000	600 000	4 000 000	14 000 000	29
Masonic Building	19-story lofts and meeting-rooms	292	23 300	423 000	1 000 000	5 701 000	17
Fire Companies' Building	25-story office-building	315	20 500	484 000	2 000 000	6 295 000	32
Walker-Lispenard Building	17-story telephone-exchange	338	20 300	330 000	1 400 000	6 270 000	22
Woolworth Building	55-story office-building	750	29 300	900 000	3 500 000	11 800 000	26
Grand Stand, Polo-Grounds	2-story grand stand	...	139 200	192 400	100 000	....	..
United States Rubber Co's Building	20-story office-building	273	10 800	226 000	750 000	2 940 000	25
Apartment-House	12-story apartment-house	139	14 300	174 000	1 000 000	2 080 000	48
Hyde Building	20-story office-building	288	14 700	315 000	1 000 000	4 500 000	22
Auerbach Candy-Factory	11-story factory	163	40 166	424 600	750 000	6 260 000	12
Title Guarantee and Trust Co's Building	8-story bank	162	12 300	112 000	1 000 000	2 240 000	45
Æolian Building	17-story lofts	252	15 700	281 000	900 000	4 125 000	22

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[*] Taken from the Report of the Bureau of Buildings of the City of New York for the Borough of Manhattan, for the year ending Dec. 31, 1911.

† These items were furnished by E. S. Hand.

The Grand Central Station, as given in the list, is only a part of the compl
terminal, being that part which contains the passenger-concourse and
waiting-rooms, restaurant and other parts that are considered necessary to
for the traffic. The cubic contents of this part total about 14 000 000 cu
Other parts of the building were erected under previous permits. Some inter
esting facts as to the complete main station are:

Cost, about	$8 000 000
Ground-area above street-level, square feet	266 [illegible]
Additional station-facilities under street, square feet	80 000
Floor-area devoted to station-purposes, square feet	1 188 000
Cubic contents, about, cubic feet	32 857 500
Steel used in construction, tons	55 797
Weight of largest girder used, tons	9

The Cost of Reinforced-Concrete Buildings.*† In estimating the co
a building by CUBICAL CONTENTS or by AREAS OF FLOORS the shape of the bu
ing in plan should be taken into consideration. A long, narrow building
cost more per cubic or square foot than one more nearly square in plan; a
computing costs by the cubic-foot or square-foot unit prices these cond
as well as the judgment and experience of the architect or engineer who m
the estimates affect the accuracy of the results. The following notes qu
from data furnished by the architects and engineers of the buildings menti
include useful information relating to costs of some contemporaneous reinfo
concrete buildings of different types, erected in Philadelphia and vicinity.

(1) "A reinforced-concrete building of the FACTORY-TYPE, erected (191
in the City of Philadelphia. It is a concrete cage, with no brick veneer,
stories in height, no basement, size, 60 by 159 ft, stair-shafts and ele
shafts projecting beyond the building; cubical contents, 603 000 cu ft.
cost, without equipment, is 7½ cts per cu ft. Drainage is included in
price, but no plumbing, heating, lighting or elevators. The total floor-ar
the building is 40 140 sq ft and the cost per square foot is $1.14½. This is
according to the building laws of Philadelphia.

(2) "A MILL-CONSTRUCTED BUILDING, about the same size as buil
recently erected in a manufacturing town forty miles from Philadelphia.
four stories in height and has a part-basement, a wing 30 by 40 ft, and
story boiler-room and engine-room. The total cubical contents are
cu ft, and the cost, 6⅗ cts per cu ft. The total floor-area is 37 900 sq ft,
cost, $0.85⅕ per sq ft. This is without power, heat, or light. There are
plumbing-fixtures in this building.

"In comparing the costs of the two buildings, it must be borne in mind
one is located forty miles from Philadelphia, and was not erected unde
rigid building laws that are in force there. It is usually possible to
building of any type at less expense outside of Philadelphia than in that
and this can probably be said of any city where there are no state building

(3) "A MILL-CONSTRUCTED BUILDING, three stories in height, erected in
in Camden, N. J., and having 575 044 cu ft. It cost 7 cts per cu ft. It
38 912 sq ft of floor-area, at a cost of $1.04 per sq ft. This price is wi
power, heat, light, or elevators, but includes some plumbing.

(4) "The new municipal REPAIR-SHOP of the City of Philadelphia. Thi
reinforced-concrete building with brick veneer of an ornamental type, and
9⅛ cts per cu ft for 1 080 591 cu ft or $1.74 per sq ft for 57 323 sq ft of

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* Valuable data on this subject has been furnished the Editor by Ballinger & Pe
the architects and engineers of the five reinforced-concrete buildings described.

† See, also, page 153-.

s is without plumbing, power, heat, light, or elevators. The
ost per square foot for this building is due to the fact that the
akes up a considerable portion of the building, so that a floor
e the crane is placed, and the floor-area accordingly reduced.
/ building for the AUTOMOBILE CLUB of Philadelphia. This is
uilding, of reinforced-concrete cage-construction, and contains
at a cost of 10$\frac{3}{5}$ cts per cu ft. The total floor-area is 90 602
.54 per sq ft. This is without power, heat, light, or any equip-
des plumbing. The shape of this building favors economy of
it is nearly square in plan."
ıp the conclusions arrived at in regard to the average costs of
ngs, E. G. Perrot states * that the cost can best be considered
ıem under three general heads:
es and manufactories. Cost, from 8 to 11 cts per cu ft.
l loft-buildings. Cost, from 11 to 17 cts per cu ft.
ous buildings, such as school-houses, hospitals, etc. Cost from
cu ft.

lls and Factories Built on the Slow-Burning Principle.
g to total and unit costs of buildings of this type, see Chapter
t to 810.

f Cost of Items of Construction in Fire-Proof Buildings

on the following six pages show, on pages 1540 to 1545, the
COSTS of fire-proof buildings among the different materials and
nstruction, the data having been furnished the compiler by
uilders in the cities mentioned in the tables. Each column of
les gives the data for an individual building, except the values
City, in the second, third and fifth columns, which show the
arge number of buildings. The tables on the first four pages
ldings approximating closely the standard specifications of the
of Fire Underwriters. The tables show that the foundations
, the only parts little damaged in conflagrations, represent,
nly 25% of the entire sound value of a building. For example,
the first four pages, the average cost of all the foundations is
verage cost of the steel frames is 17.88%. The tables show,
44 and 1545 the percentages of cost of the classified items of
ight buildings damaged by the Baltimore conflagration (1904),
hese eight buildings being given in the last column.

ative Costs of Reinforced Concrete Buildings," by E. G. Perrot, in
National Association of Cement Users, Vol. V, 1909.

the first four pages were compiled by F. J. T. Stewart, Continental
y, and those on the last two pages by the Baltimore Committee of
l of Fire Underwriters. All are reproduced, by permission, from J. K
vention and Fire Protection. Those parts of the Baltimore tables
portion of fire-damage to sound value of the various items have been
icle of the Pocket-Book deals more especially with original costs.

Table Showing Proportion of Value in the Various Items of Construction of Fire-Proof Buildings

The figures opposite each item represent percentages of total cost of building

Classified construction	New York					Chicago						Baltimore							
	Ofcs	Ofcs	Hotel	Ofcs	W.H.	Ofcs	Ofcs	Ofcs	Ofcs	Merc	Merc	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs
Height, in stories	10	...	....	12	...	...	...	...	...	...	...	11	12	7	14	9	8	10	7
Cost, cents per cubic foot	...	...	....	40	...	30.3	34.8	32.3	...	14	23.6	...	...	...	...	...	...	...	...
Foundations	...	...	....	...	...	*4.4*	*2.34*	*5.67*	...	*6.00*	*10.74*	*5.62*	*4.37*	*7.25*	...	...	...	...	*4.3*
Excavations of back-filling	...	...	....	...	...	...	...	...	...	...	...	1.22	1.25	2.07	...	...	...	...	4.3
Shoring banks, etc.	...	...	....	...	...	...	...	...	...	...	...	...	1.06	...	...	...	...	...	...
Foundation-footings and concrete	...	...	....	...	...	...	...	...	...	...	...	4.4	0.74	5	...	...	...	...	...
Rubble-stone and granite pier-caps	...	...	....	...	...	...	...	...	...	...	...	...	0.65	...	...	...	...	...	...
Sidewalks and curbs	...	...	....	...	...	...	...	...	...	...	...	...	0.13	0.18	...	...	...	...	...
Steel frame	*24.0*	*18.4*	*16.0*	*22.50*	*36.5*	*19.70*	*14.0*	*11.86*	*10.6*	*27.6*	*38.66*	*13.6*	*14.54*	*10.52*	*9.9*	*9*	*10.1*	*11.4*	*17.5*
Material	...	...	....	...	...	...	...	...	...	...	...	...	11.79	...	...	...	...	...	...
Erection	...	...	....	...	...	...	...	...	...	...	...	...	1.7	...	...	...	...	...	...
Shop-drawings	...	...	....	...	...	...	...	...	...	...	...	...	0.49	...	...	...	...	...	...
Painting	...	...	....	...	...	...	...	...	...	...	...	...	0.24	...	...	...	...	...	...
Teaming	...	...	....	...	...	...	...	...	...	...	...	...	0.32	...	...	...	...	...	...
Masonwork	*26.96*	*26.2*	*31.5*	*20.10*	*32.5*	*26.31*	*27.96*	*22.3*	*38.5*	*27.6*	*44.79*	*23.76*	*28.5*	*34.15*	*30.15*	*31.31*	*37.2*	*20.0*	*35.5*
Brick, common	...	...	....	...	...	...	...	...	...	...	...	10.1	6.59	11.21	...	...	...	8.98	13.5
Brick, faced or pressed	...	...	....	...	...	...	...	...	...	...	...	...	1.82	1.26	...	...	...	...	...
Brick, enameled	...	...	....	...	...	...	...	...	...	...	...	...	0.86	...	...	...	...	...	...
Brick, cleaning and pointing	...	...	....	...	...	...	...	...	...	...	...	...	0.21	...	...	...	...	...	...
Terra-cotta	1.41	...	....	2.06	...	10.04	...	5.76	...	...	...	4.34	7.61	5.07	3.9	2.5	...	3.5	...
Stone	2.75	...	....	1.50	...	...	10.30	...	...	...	...	2.55	4.22	9.34	4.05	7.4	27.3	8.7	9.4
Marble	...	...	....	10.90	...	...	...	...	...	...	...	...	...	...	....	0.83	...	...	...
Wall-lining or furring	...	...	....	...	...	...	...	...	...	...	...	0.77	...	...	...	1.8	...	...	...
Floor-arches, roof, etc.	...	...	....	...	...	11.06	9.16	5.8	...	...	22.08	3.87	4.71	3.68	6.8	6.6	3.4	3.6	6.6
Cinder-concrete filling over arches	...	...	....	...	...	1.06	1.30	1.24	...	...	0.47	0.57	0.58	0.49	...	...	...	...	6.
Partitions	...	...	....	...	...	...	...	...	...	...	...	1.86	1.85	3.1	...	...	...	...	...
Partitions, cleaning and wrecking	...	...	....	...	...	...	...	...	...	...	...	...	...	...	7.6	...	...	5.16	...
Safety-deposit vaults	...	...	....	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Miscellaneous, scaffolding and																			

The figures opposite each item represent percentages of total cost of building

Classified construction	New York					Chicago						Baltimore							
Equipment	*20.65*	*21.2*	*12.0*	*20.89*	*11.25*	*24.96*	*24.54*	*25.04*	*23.2*	*20.8*	*2.26*	*19.18*	*24.54*	*15.96*	*18.69*	*26.71*	*15.23*	*21.23*	*12.69*
Elevator-plant	5.70	...	....	5.28	...	4.65	7.81	5.48	...	...	...	7.6	7.68	8.15	5	5.68	3.8	7.7	4.02
Plumbing	3.49	...	....	4.16	...	6.58	5.90	6.19	...	...	0.85	3.26	3.55	2.61	4.13	3.5	3.7	3.4	3.36
Heating-system	5.85	...	....	5.85	...	...	0.67	5.88	...	...	...	4.6	7.06	3.54	4.2	3.6	2.2	6.7	4.15
Boiler-plant	...	...	....	...	...	...	1.92	1.86	...	...	0.85	...	...	...	...	1.9	...	...	...
Lighting-system, wiring and fixtures	5.37	...	....	5.6	...	3.82	2.76	...	...	...	...	2.54	5.05	1.38	2.03	1.9	1.6	3	1.10
Dynamos, switchboards, etc.	...	...	....	...	...	...	...	3.34	...	...	...	...	...	...	...	2.18	...	...	...
Fixtures	...	...	....	...	...	...	0.32	1.24	...	...	...	0.7	0.49	...	1.05	0.65	...	...	...
Mail-chute	0.24	...	....	...	...	0.24	0.06	0.13	...	...	...	0.48	0.27	0.28	0.18	...	0.33	0.43	...
Filter-plant	...	...	....	...	...	...	...	...	...	...	...	...	0.44	...	...	...	...	...	...
Refrigerating-plant	...	...	....	...	...	...	...	...	...	...	...	...	...	...	0.93	...	...	...	...
Safes	...	...	....	...	...	...	...	...	...	...	...	...	...	...	...	1.9	...	...	...
Vault-doors and safe-doors	...	...	....	...	...	0.24	0.29	0.32	...	...	0.56	...	...	...	0.63	...	3.6	...	...
Flash-signals and indicators	...	...	....	...	...	...	...	...	...	...	...	...	...	...	0.54	...	...	...	...
Furniture	...	...	....	...	...	...	...	...	...	...	...	...	...	...	...	5.4	...	...	...
Ventilation	...	...	....	...	...	9.43	4.80	0.60	...	...	...	...	...	...	...	...	...	...	...
Trim and finish	*28.40*	*29.8*	*33.25*	*22.26*	*14.75*	*24.44*	*29.45*	*35.28*	*27.7*	*18.3*	*4.39*	*36.06*	*27.56*	*22.83*	*36.20*	*28.78*	*36.63*	*26.89*	*25.04*
Carpentry, rough	...	...	....	...	...	8.90	6.53	8.54	...	...	2.86	2.03	2.59	0.22	10.9	9.8	5.4	7.9	1.6
Carpentry, finish	20.50	...	....	10	...	...	1.88	...	...	...	...	5.27	4.31	0.51	...	...	...	...	5.7
Hardware, rough	...	...	....	...	...	0.96	1.70	1.5	...	...	...	0.32	0.29	1.7	...	0.64	0.9	1.16	1.95
Hardware, finish	...	...	....	...	...	...	...	...	...	...	...	0.74	0.88	5.1	0.83	...	...	...	...
Marble	2.83	...	....	9.85	...	5.02	4.17	7.61	...	...	0.25	6.74	7.11	1.42	10.5	5.4	10.1	6.2	2.8
Mosaic	1.25	...	....	...	...	...	...	...	...	...	...	0.14	...	1.99	...	...	1.2	...	...
Glass	...	...	....	...	...	1.02	1.32	3.37	...	...	...	1.44	9	1.13	1.4	7.5	1.4	...	0.89
Slate	...	...	....	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Plastering	3.82	...	....	2.41	...	2.97	3.68	2.35	...	...	...	2.94	3.05	2.96	2.9	4.21	3.7	2.7	1.4
Fresco	...	...	....	...	...	1.84	3.49	1.78	...	...	...	0.88	...	...	...	...	...	...	...
Paint and varnish	...	...	....	...	...	1.64	2.05	1.39	...	...	0.30	2.00	1.34	1.45	2.9	1.6	1.4	2.9	1.5
Office-partitions (wood and glass)	...	...	....	...	...	...	...	...	...	...	...	2.65	...	...	...	...	...	...	...
Ornamental iron	...	...	....	...	...	1.09	2.58	6.19	...	...	...	9.70	5.87	4.62	6.4	5.0	10.5	5.5	6.8
Skylights and sheet metal	...	...	....	...	...	1.00	2.05	2.55	...	...	0.98	0.71	1.22	1.54	0.37	0.63	0.83	0.53	2.4
Office-grill	...	...	....	...	...	...	...	...	...	...	...	0.5	...	0.19	...	...	1.2	...	...
General expenses	...	*4.4*	*7.70*	...	*5.00*	...	...	...	...	...	...	*1.83*	*1.92*	...	*5.17*	*4.31*	*0.36*	*11.2*	*6.94*

Table (Continued) Showing Proportion of Value in the Various Items of Construction of Fire-Proof Buildings

The figures opposite each item represent percentages of total cost of building

Classified construction	Boston																	St. Louis	
	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Merc	Merc	Merc	Merc	Merc	Sch'l	Stg	Merc	Merc	Ofcs	Ofcs
Height, in stories	9	11	10	10	11	11	11	11	7	4	6	3	5	3	7	5	6	12	12
Cost, cents per cubic foot	24	...	35	49	67	41.8	45	34	...	...	21.5	22	29	29.6	...	...	...	...	...
Foundations	*10.6*	*5.13*	*4.32*	*7.5*	*4.42*	*5.8*	...	...	*14.2*	*14*	...	*11.5*	*13.5*	*7.4*	...	*24.5*	...	*5*	*7*
Excavations and back-filling	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Shoring banks, etc.	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Foundation-footings and concrete	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Rubble-stone and granite pier-caps	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Sidewalks and curbs	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Steel frame	*14.4*	*27.4*	*22.5*	*15*	*25.7*	*21.8*	*21.4*	*24.6*	*8.25*	*2.2*	*22.1*	*19.32*	*14.65*	*21.2*	*21.6*	*14.3*	*7.95*	*15.0*	*15.0*
Material	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Erection	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Shop-drawings	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Painting	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Teaming	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Masonwork	*41.4*	*15.0*	*32.23*	*41.6*	*33.60*	*27.57*	*34.85*	*26*	*33.65*	*31.45*	*22.88*	*30.6*	*17.42*	*30.46*	*68.62*	*26*	*38.6*	*20.0*	*25.0*
Brick, common	27.5	8.3	18.9	15	19.7	17.75	17.7	16.9	29.2	30	16.8	24	9.75	20.1	64	23.8	...	...	...
Brick, face or pressed	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Brick, enameled	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Brick, cleaning and pointing	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Terra-cotta	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Stone	7.1	1.75	8.17	16.6	5.25	3.55	10.4	4.54	4.45	1.45	3.3	6.6	6.95	6.6	4.62	2.14	...	...	...
Marble	5.8	4.95	5.16	10	8.65	6.27	6.75	4.56	...	...	2.78	...	0.72	3.76	...	0.08	...	...	...
Wall-lining or furring	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Floor-arches, roof, etc.	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Cinder-concrete filling over arches	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

Classified construction	Boston																	St. Louis	
Equipment	*9.48*	*30.3*	*18.05*	*15.99*	*24.45*	*20.12*	*23.02*	*23.53*	*8.98*	*23.9*	*18.71*	*12.3*	*11.84*	*16.25*	*2.46*	*10.65*	*19.09*	*17*	*16*
Elevator-plant	3.29	12.4	3.5	4.66	3.92	5.56	4.3	7.1	7.65	1.54	...	5.07	2.44	2.39	...	4.25	6.45	...	...
Plumbing	1.74	4.75	3.4	4	4.8	5.45	2.92	5.47	0.51	1.85	3.68	2.9	5.07	4.32	...	1.25	1.5	...	...
Heating-system	2.71	8.4	4.83	5.33	2.88	2.48	8.46	7.8	...	9.85	5.5	3.34	2.34	7.64	1.54	1.36	2.52	...	...
Boiler-plant	...	...	1.4	...	5.68	2.37	...	...	...	10.4	7.15	...	...	...	...	1.07	...	...	...
Lighting-system, wiring and fixtures	1.74	4.75	3.79	2.00	1.62	4.26	7.34	3.16	...	0.26	2.38	1.03	1.99	1.9	0.92	1.5	7.2	...	...
Dynamos, switchboards, etc.	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Fixtures	...	...	1.13	...	5.55	...	...	...	0.825	...	...	...	...	...	...	1.22	1.42	...	...
Mail-chute	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Filter-plant	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Refrigerating-plant	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Safes	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Vault-doors and safe-doors	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Flash-signals and indicators	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Furniture	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Ventilation	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Trim and finish	*20.87*	*22.11*	*22.4*	*19.34*	*12.16*	*24.73*	*21.30*	*26.11*	*34.18*	*28.27*	*36.32*	*26.36*	*41.35*	*24.92*	*7.24*	*24.70*	*34.57*	*33*	*27*
Carpentry, rough	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Carpentry, finish	16.6	14.3	13.4	13.2	6.54	15.75	13.6	18.9	31.2	24.4	16.8	17.5	29	15.1	...	19.9	30.8	...	...
Hardware, rough	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Hardware, finish	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Marble	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Mosaic	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Glass	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Slate	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Plastering	3.32	5.5	3.3	2.66	1.93	5.2	2.74	3.55	0.572	...	12.6	2.0	6.7	4.3	1.54	0.51	...	...	...
Fresco	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Paint and varnish	2.9	1.65	4	2.33	2.63	2.6	3.74	2.68	1.14	1.58	5.4	3.26	3.6	2.8	0.30	2.86	2.34	...	...
Office-partitions (wood and glass)	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Ornamental iron	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Skylights and sheet metal	2.05	0.66	1.7	1.15	1.06	1.18	1.22	0.986	1.27	2.29	1.52	3.6	2.05	2.72	5.4	1.43	1.43	...	...
Office-grill	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
General expenses	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	*10*	*10*

Table Showing Proportion of Value in the Various Items of Construction in Eight Fire-Proof Office-Buildings Erected in Baltimore, Md., Before the Conflagration. The figures opposite each item represent percentages of total cost of building

Classified construction	Union trust Building	Calvert Building	Herald Building	Continental Building	Equitable Building	Mer Nat Bank Building	Maryland Trust Building	C & P Tel Co Building	Average for eight buildings
Foundations	*5.62*	*4.37*	*7.25*	....	....	....	....	*4.3*	*5.5*
Excavations and back-filling	1.22	1.25	2.07	....	....	....	....	4.3	2.21
Shoring banks and holding adjacent property	....	1.6	....	....	....	....	....	....	1.6
Foundation-footings and concrete	4.4	0.74	5	....	....	....	....	....	3.38
Rubble-stone and granite pier-caps	....	0.65	....	....	....	....	....	....	0.65
Sidewalks and curbs	....	0.13	0.18	....	....	....	....	....	0.155
Steel frame	*13.6*	*14.54*	*19.52*	*9.9*	*9.00*	*10.1*	*11.4*	*17.5*	*13.195*
Material	....	11.79	....	....	....	....	....	....	11.79
Erection	....	1.7	....	....	....	....	....	....	1.7
Shop-drawings	....	0.49	....	....	....	....	....	....	0.49
Painting	....	0.24	....	....	....	....	....	....	0.24
Teaming	....	0.32	....	....	....	....	....	....	0.32
Masonwork	*23.76*	*28.50*	*34.15*	*30.15*	*31.31*	*37.2*	*20.04*	*35.5*	*31.31*
Brick, common	10.1	6.59	11.21	....	....	....	8.98	13.5	8.25
Brick, face or pressed	....	1.82	1.26	....	....	....	....	....	1.54
Brick, enameled	....	0.86	....	....	....	....	....	....	0.86
Brick, cleaning and pointing	....	0.21	....	....	....	....	....	....	0.21
Terra-cotta	4.34	7.61	5.07	3.9	2.5	....	3.5	....	4.48
Stone	2.55	4.22	9.34	4.05	7.4	27.3	8.7	9.4	9.12
Marble	....	....	....	....	0.83	....	....	....	0.83
Wall-lining or furring	0.77	....	....	....	1.8	....	....	....	1.28
Floor-arches, roof, etc.	3.87	4.73	3.68	6.8	6.6	3.4	3.6	6.6	4.91
Cinder-concrete filling over floor-arches	0.57	0.58	0.49	....	....	....	....	6.0	1.91
Partitions	1.56	1.88	3.1	....	....	....	....	....	2.18
itions, cleaning and wrecking	....	....	....	....	....	....	....	....	....
Safety-deposit vaults	....	....	....	7.6	....	....	5.16	....	6.38

er-plant					1.9				1.9
Lighting-system, wiring and fixtures	2.54	5.05	1.38	2.03	1.9	1.6	3	1.16	2.33
Dynamos, switchboards, etc.					2.18				2.18
Fixtures	0.7	0.49		1.05	0.65				0.72
Mail-chute	0.48	0.27	0.28	0.18		0.33	0.43		0.32
Filter-plant		0.44							0.44
Refrigerating-plant				0.93					0.93
Safes					1.9				1.9
Vault-doors and safe-doors				0.63		3.6			2.11
Flash-signals and indicators				0.54					0.54
Turkish baths					0.68				0.68
Furniture					5.4				5.4
Piping, high pressure					0.70				0.70
Trim and finish	*36.06*	*27.56*	*22.83*	*36.20*	*28.78*	*36.63*	*26.89*	*25.04*	*20.99*
Carpentry, rough	2.03	2.59	0.22	10.9	9.8	5.4	7.9	1.6	5.05
Carpentry, finish	5.27	4.31	0.51					5.7	3.94
Hardware, rough	0.32	0.29	1.7		0.64	0.9	1.16	1.95	0.99
Hardware, finish	0.74	0.88	5.1	0.83					1.88
Marble	6.74	7.11	1.42	10.5	5.4	10.1	6.2	2.8	6.28
Mosaic	0.14		1.99			1.2			1.11
Glass	1.44	9	1.13	1.4	7.5	1.4		0.89	1.23
Slate									
Plastering	2.94	3.05	2.96	2.9	4.21	3.7	2.7	1.4	2.98
Fresco	0.88								0.88
Paint and varnish	2.00	1.34	1.45	2.9	1.6	1.4	2.9	1.5	1.88
Office-partitions (wood and glass)	2.65								2.65
Ornamental iron	9.70	5.87	4.62	6.4	5.0	10.5	5.5	6.8	6.79
Skylights and sheet metal	0.71	1.22	1.54	0.37	0.63	0.83	0.53	2.4	1.02
Office-grill	0.5		0.19			1.2			0.63
General expenses	*1.83*	*1.92*		*5.17*	*4.31*	*0.36*	*11.2*	*6.94*	*4.53*
Miscellaneous	1.83	1.92		0.27	0.67	0.36	10	1.94	2.43
Architects' fees, etc.				4.9	1.82			5.0	3.91
Permits, bond, survey, insurance, etc.							1		1.2
Cleaning out debris after fire					1.82		0.2		1.82

Cost of Different Kinds of Work per Cubic Foot of

Some estimates * have been made by F. W. Fitzpatrick showin
tionate COST OF THE DIFFERENT BRANCHES OF WORK which go t
completed building. Believing that these data will be found usef
up approximate estimates, Mr. Kidder obtained permission to use
Pocket-Book. The following figures represent the actual cost of
OFFICE-BUILDING, 60 by 130 ft in plan, built in the Middle West
fire-proof structure, with two street-fronts faced with granite and
pile foundation.

Kind of work	Per cubic foot of entire building, cents	Kind of work
Foundations	1¾	Heating
Steel framing	2½	Plumbing
Granite and all masonry	11⅙	Elevators
Cornice, roofs and skylights	⅔	Stairs, scenic structural framing, "making ends meet," lamp-fixtures, etc. What might be called a fair amount for "contingencies" in such a building, including lesser items not mentioned here but grouped together
Fire-proof floors	⅔	
Partitions, tile	⅗	
All plastering and stucco	1¼	
Elevator-fronts and all ornamental metalwork	2	
Marblework	3⅙	
Hardware	2/15	
Joiners' work	1⅙	
Glass	5/12	Architect's fee
Painting and varnishing	7/10	
Electric wiring	⅔	Total

The Chicago post-office building, containing 12 000 000 cu ft and
mental character and finish, cost, in some of its items, as follows:

Kind of work	Per cubic foot of entire building, cents	Kind of work
Foundations	1¾	Ornamental metalwork
Steel framing	2½	Marble
Granite and masonry	13½	Plumbing
Fire-proof floors	⅔	Heating
Plaster, plain and ornamental	1⅞	

It will be noticed that the relative cost of several of these items is
as in the office-building. The total cost of this building was

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* Published in "Fireproof," March, 1903.

Cost of Buildings per Square Foot

ldings of Large Area, such as exposition-buildings, etc., lmost as accurately by the square foot of ground covered as f building, as there are few or no interior partitions, and usu- r interior finish.

Buildings. "Roughly speaking, the cost of one-story iron , complete, is, for sheds and storage-houses, from 40 to 60 cts d, and for such buildings as machine-shops, foundries, and , that are provided with traveling cranes, the cost is from 60 of ground covered." *

l. For estimates of cost of structural steel for buildings, 527.

rick Mills and Warehouses. See Chapter XXII, pages

dings. The cost † of the World's Fair buildings (Chicago, oot of ground covered, including sculpture and decoration,

and Liberal Arts Building	$1.39
Building	1.08
lding	1.69
ll	2.12
uilding	1.44
Building	9.18
Building	1.41
ning Building	1.04
ling	2.35
ling	0.75

ures for the St. Louis Exposition (1904). The following by Isaac S. Taylor, at that time Director of Works, of the

ng	Dimensions, ft	Area, acres	Total cost	Cost, sq ft
	161×346	1.42	$967 833.90	$5.45
, each	144×423	3.14		
ex	106×150	0.41	39 388.99	2.48
ling	200×736	3.86	328 980.00	2.23
eries	136×136	0.42	45 000.00	2.43
rgy	525×750	9.08	488 848.50	1.24
	525×750	8.80	471 820.95	1.20
ial Economy	525×758	7.70	323 950.75	0.81
	525×1 200	13.47	711 510.00	1.13
	525×758	6.67	408 531.57	1.03
	525×1 200	10.28	704 067.96	1.12
	525×1 000	9.48	509 110.50	0.97
uel	301×326⅔	2.25	135 480.00	1.38
	525×1 300	15.70	674 853.42	0.99
	374×782	5.42	225 342.27	0.77
	500×1 600	18.62	520 491.07	0.58
l Game	300×600	4.07	168 883.38	0.94
	195 in diameter, *exclusive of annex*	*1.09*	215 899.00	....

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† *Given by E. C. Shankland, chief engineer.*

by the Committee on Public Buildings and Grounds of the U
a list of sites and plans for public buildings, giving data of m
to the cost of public buildings, their cubical contents and
foot, including buildings erected from 1816 to 1910. "As a
have cost more per cubic foot than private buildings, so th
always be used as a guide, except for government buildings.

**Unit Prices per Cubic Foot for Recent Governmen
Same Type.**‡ The data included in the following paragrap
buildings recently erected or in process of construction in
certain FIXED TYPES and in different parts of the United Sta
are post-office buildings and the location, brief description
struction, ground-area covered, cubical contents and com
cubic foot are given. The buildings are grouped under five
the VARIATIONS IN COSTS PER CUBIC FOOT of similar or identic
type, located in different sections of the country, are show
five types is a list of buildings of various sizes and descri
variations in the cubic-foot rates. The conclusions arrived
at the end of the lists, include a table which shows what i
office of the Supervising Architect to be a fair DIFFERENCE IN
OF THE SAME TYPE in different sections of the United States
also, by this office, that the method of estimating the cost
CUBIC-FOOT UNIT PRICE is productive of very uncertain re
there are many variable conditions entering into the constr
located in different localities. The principal items affecting
types of buildings are:

(1) Labor; rates and efficiency.
(2) Materials; quality and freight-rates.
(3) Season; time of year when building is constructed.
(4) Contractors; finances, ability, equipment, overhead e
of profit desired.

as to supply-centers, distance from railroads, and facilities for als.

n Unit Costs of Identical Buildings in Different Localities. npare the costs of identical buildings, with slight modification ing are given as examples, to show the variance in different

t-office buildings at Grenada, Miss., Bennettsville, S. C., Cov- nd Burlington, N. J.

Main building, two stories and basement; rear projection, one ent; non-fire-proof construction throughout; brick facing; stone ornice; slate-covered gable roof, with dormers over two-story , composition roof over one-story portion.

Area and contents	
	3 825 sq ft
ts	138 210 cu ft

Rate per cubic foot		
Location	Non-fire-proof	First floor, fire-proof
	$0.322	$0.327
n.	0.315	0.324
S. C.	0.304	0.309
J.	0.293	0.298

t-office buildings at Winchester, Tenn., McPherson, Kan., and

Main building, two stories; rear projection, one story; partly ment; non-fire-proof construction throughout; brick facing; den cornice and pilasters at front entrance; slate-covered gable rs over two-story portion, and flat, composition roof over one-

Area and contents	
	3 825 sq ft
ts	138 210 cu ft

Rate per cubic foot		
Location	Non-fire-proof	First floor, fire-proof
n.	$0.344	$0.350
n.	0.346	0.351
	0.332	0.337

Type 3. Post-office buildings at Cookeville, Tenn., and Jackson, Ky.

Description. Three-story-and-basement building; stone-faced to top course over water-table; selected, common-brick facing and ornamental ter cotta trim; composition and slate roof and non-fire-proof construction, exc the first floor.

Area and contents	
Ground-area	4 942 sq ft
Cubical contents	290 300 cu ft
Rate per cubic foot	
Cookeville, Tenn.	$0.275
Jackson, Ky.	0.269

Type 4. Post-office buildings at Garden City, Kan., and Lake City, Minn (identical buildings).

Description. One-story-and-basement, brick-faced building, with stone water-table course and trimmings and ornamental terra-cotta cornice, architrave and parapet-coping; non-fire-proof construction, except the first floor; composition roof.

Area and contents	
Ground-area	3 888 sq ft
Cubical contents	141 456 cu ft
Rate per cubic foot	
Garden City, Kan.	$0.405
Lake City, Minn.	0.341

Type 5. Post-office buildings at Abilene, Kan., and Bellefontaine, Ohio.

Description. One story and basement; stone facing; granite steps, etc.; tin roof; fire-proof construction, except roof.

Area and contents	
Ground-area	5 000 sq ft
Cubical contents	183 000 cu ft
Rate per cubic foot	
Abilene, Kan.	$0.359
Bellefontaine, Ohio	0.367

Buildings of Various Sizes and Descriptions. The following list is for buil ings of various sizes and descriptions throughout the country with the variance in the cubic-foot rate.

t-office building at New Rochelle, N. Y.

scription. This building is of an irregular plan; two-story and basement; r pavilion; sides and rear one-story and basement; clearstory over work-; stone facing to first-floor level; brick facing above this point, with terra-trim and cornice; composition roof; fire-proof construction.

Ground-area	7 512 sq ft
Cubical contents	258 900 cu ft
Rate per cubic foot	$0.259

ost-office building at Mobile, Ala.

escription. Front portion, two stories, and rear portion, one story over kroom. Only a small portion of basement excavated for heating-plant. n building faced with limestone and rear second story portion with orna-tal terra-cotta. Fire-proof construction; long and short spans, and con-e joists with terra-cotta fillers; copper deck and Spanish-tile roofs.

Ground-area	18 054 sq ft
Cubical contents	670 476 cu ft
Rate per cubic foot	$0.341

ost-office building at Muskogee, Okla.

escription. A four-story-and-basement building. Granite to the first-r line, stone-faced above (except in interior court, which is brick); terra-a cresting at roof; copper roofing and fire-proof construction throughout. h standard types of concrete and terra-cotta floor-construction. Monu-tal in design. Corinthian colonnade at entrance. Eight heavy bronze p-standards. Six flights of marble stairs. Entire lobby of marble, and y ornamental plaster-work in lobby and court-room.

Ground-area	20 400 sq ft
Cubical contents	1 326 612 cu ft
Rate per cubic foot	$0.43

ost-office building at New Bedford, Mass.

escription. One story, basement and mezzanine with clearstory over tral portion; granite facing, except clearstory, which is faced with terra-ta; main roof of composition; clearstory roof of copper; fire-proof con-ction.

Ground-area	27 750 sq ft
Cubical contents	1 080 690 cu ft
Rate per cubic foot	$0.323

ost-office building at Newark, Ohio.

escription. Two-story, basement and unfinished attic. The workroom ex-ds through two stories. Offices in second story over balance of building. -proof construction throughout. Terra-cotta floors, ceilings, roofs, parti-s, furring, etc. Exterior faced with pink granite to the first-floor level and h white marble above, including cornice, parapet, etc. Flat tin roof; bronze es at first and second-story windows on front of building. Cast-iron grilles first-story and basement-windows on sides and rear; bronze-faced post-e screens, desks, revolving doors, vestibules, etc., and drawn-bronze covered es, window-frames, doors, etc., in lobby. Caen-stone cornice and coffered g in lobby. Bronze and marble stairs to second story,

Ground-area	6 912 sq
Cubical contents	369 640 cu
Rate per cubic foot	$0.487

Post-office building at Minot, N. D.

Description. Three-story-and-basement building; fire-proof,
which is plank on steel beams; stone facing to second-story wind
facing above, with stone cornice, parapet-coping, etc.

Ground-area	6 700 sq
Cubical contents	427 300 cu
Rate per cubic foot	$0.328

Post-office building at McAlester, Okla.

Description. Three stories and basement; fire-proof, excep
cotta floors, etc.; suspended ceilings; stone facing to second-floo
facing above, with stone trim; cornice and balustrade; tin roof.

Ground-area	7 482 sq
Cubical contents	394 765 cu
Rate per cubic foot	$0.38

Post-office building at North Tonawanda, N. Y.

Description. The building has two stories and basement;
first-floor line; brick-faced above with stone trimming and slate
construction to and including the second floor.

Ground-area	5 475 sq
Cubical contents	276 320 cu
Rate per cubic foot	$0.289

Conclusions Regarding Variations in Unit Costs. In
unit costs, the APPROACH-WORK, such as walks, platforms, terra
cluded. This, in some cases, is quite expensive, and is generally
of the entire cost of the building. In federal buildings, there are
ments not met with in the ordinary mercantile buildings, and
character of the building necessitates all materials, workmanship
tion to be of the very best in each case. This is guaranteed by
fications, long-time guarantees for several items of the work,
government inspection. The office of the supervising archit
mined that the RELATIVE INCREASE in cost of buildings througho
over the cost in the Mississippi Valley district is about as follo
Mississippi Valley district, as a BASE, at 100%, and the labor a
ditions which prevailed in October, 1914.

	Pe
Mississippi Valley district	
New England (except Maine)	
Maine	
Southern States	
Northwest Mountain district	
Southwest Mountain district	
Pacific Coast	

g of districts, the Mississippi Valley district is intended to
States as far east as Ohio and Pennsylvania, and the states,
ing on the western bank of the Mississippi River. This is
rt of the country in which the LOWEST PRICES have been ob-
r districts represent the approximate greater cost for buildings
Mississippi Valley or Middle States, and is intended to repre-
NCE IN COST AT ANY TIME; but is not intended to represent
ost at different periods.

Variation in Cost of Buildings of Identical Area and
following notes are taken from photographs of drawings and
panying them.* The drawings were for a Post-Office build-
ie, Wis. This building contains 4 770 sq ft of ground-area,
ontents are 147 570 cu ft. The contract was awarded (1913)
the rate of $0.308 per cu ft. It is a one-story-and-basement
ith brick, with stone water-table, brick parapet and tin and
The first floor, only, is fire-proof. Proposals were opened
-Office building at Uvalde, Tex. This building, except for
fications, is as nearly like the Menomonie building as it is
t without using the same drawings. The ground-area of the
4 672 sq ft and the cubical contents, 151 875 cu ft. The work
the approaches is practically the same as that at Menomonie.
had been erected in the same town, it does not appear that
been any difference in the costs, but the lowest proposal
valde building was $56 400, or at the rate of $0.371 per cu ft.
the amounts for these two buildings further illustrates the
y universal application of the cubic-foot rate in determining
ings, and also shows that the difference in cost of construction
ferent sections of the country varies considerably.

Foot of Some Important Federal Buildings. The follow-
ntain additional unit costs and other data for public buildings.

Cubic Foot of Some of the Larger Public Buildings

Location and building	Cost per cubic foot, cents
., Custom-House (completed 1908)	74
Post-Office, Custom-House and Court-House	68
al., New Post-Office and Court-House (completed	66
v Mint (completed 1905)	65
al., Subtreasury Building (estimated)	60
new Custom-House (completed 1908)	55
., Senate Office-Building	50
Jtah, Post-Office (completed 1905)	47
., new Post-Office (completed 1906)	46
, new Mint (completed 1901)	45
., National Museum Building	43
., Agricultural Buildings (portions completed)	40
., House Office-Building	36

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phs of plans, elevations and sections, together with many others, and
nations and data, were furnished the editor by J. W. Ginder, Super-
nputing Division, Office of the Supervising Architect, by permission
the Supervising Architect, and have been of great assistance in
es on the costs of buildings.

Cost per Cubic Foot and per Square Foot of Some New

Location	Facing	Cost	Contents, cu ft	Are sq
Bangor, Me.	Granite	$271 297	793 720	15 6
Augusta, Ga.	Marble	288 800	576 000	11 0
South Chicago, Ill.	Stone	132 702	377 668	11 0
Long Branch, N. J.	Limestone	95 200	256 210	6
Plymouth, Mass.	Brick	81 532	256 210	6
Piqua, Ohio	Limestone	116 689	448 300	9 9
New Bedford, Mass.	Granite	295 051	1 080 000	21 7

Depreciation of Buildings †

Discounts from Values of New Buildings. The figur ceding pages are for new buildings. To ascertain their val sequent to their erection, a discount from the value when as follows:

Brick, occupied by owner. . . . .
Brick, occupied by tenant. . . . .
Frame, occupied by owner. . . . .
Frame, occupied by tenant. . . . .

If built of long-leaf yellow pine, or of spruce from the N add from 20 to 30%, or if of short-leaf yellow pine, add fro values. If of redwood or cedar from the Pacific Coast, these estimates, which are for white pine or white pine with These figures for depreciation are to include buildings in wh have been made. If extraordinary repairs have been made, not be so heavy. Good judgment must be used in estim depreciation in buildings.

The Depreciation of Mill-Buildings. The annual de building of slow-burning construction varies from 1 to preciation of a reinforced-concrete factory-building is r since it is confined entirely to such details as windows, doo

The Wear and Tear of Building Materials. At the t of the Fire Underwriters' Association of the Northwest, September, 1879, Mr. A. W. Spalding read a paper on th building materials and tabulated the results of his investig

	Frame dwelling		Brick dwelling (shingle roof)		Frame store		Brick store (shingle roof)	
building	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %
....	....	......	75	1⅓	....	......	66	1½
....	20	5	30	3⅓	16	6	30	3⅓
utside...	5	20	7	14	5	20	6	16
nside....	7	14	7	14	5	20	6	16
....	16	6	16	6	16	6	16	6
....	40	2½	40	2½	30	3⅓	40	2½
arding..	30	3⅓	....	......	30	3⅓	....	......
....	50	2	50	2	40	2½	50	2
....	20	5	20	5	13	8	13	8
plete....	30	3⅓	30	3⅓	25	4	30	3⅓
omplete.	30	3⅓	30	3⅓	25	4	30	3⅓
newels...	30	3⅓	30	3⅓	20	5	20	5
....	40	2½	40	2½	30	3⅓	30	3⅓
ds.......	30	3⅓	30	3⅓	30	3⅓	30	3⅓
rdware.	20	5	20	5	13	8	13	8
porches	20	5	20	5	20	5	20	5
nds.....	16	6	16	6	16	6	16	6
irst-floor	25	4	40	2½	25	4	30	3⅓
lumber.	50	2	75	1⅓	40	2½	66	1½

res represent the averages deduced from the replies made by eighty-
tent builders unconnected with fire-insurance companies in twenty-
nd towns of the eleven Western States.

QUANTITY SYSTEM OF ESTIMATING*

on of the System. The QUANTITY SYSTEM is not, as some persons
d, merely the taking off of a list of items by one person for some
s use. It means the careful measurement by a disinterested expert
ned in this kind of work, that is, a QUANTITY SURVEYOR. This
ceeds in a manner different from that of the average contractor.
ertain recognized rules in taking off quantities, abstracting and
view to eliminating errors. He uses also certain uniform standards
ents and expressions. His checking and rechecking methods to
cy must be studied to be appreciated by those to whom the quan-
s unknown. A record is made of every item, however small, hav-
-value. These items are classified and arranged, each under its
or department, in methodical order. Guess-work methods are
the quantity surveyor, while his accuracy and attention to even

---

tity system of estimating has been systematically advocated since 1891.
ed much attention among contractors, architects and engineers as a direct
ng-continued activity of those who recommend it. In course of time this
nating must be adopted, as it stands for a square deal between owner and
he movement in aid of this work is purely a voluntary one, an honest
about *a betterment of the existing conditions in estimating* and contracting.
tion may be obtained from G. A. Wright, 354 Pine Street, San Francisco,
of the present movement to introduce the system in the United States.

small details is worthy of comment. Every bidder figur
surveyor's quantities furnished to each one, with the pl
free of cost. The surveyor who does this work is a profe
the engineer or the architect. He should, in fact, have, a
experience in these professions, and in addition, a practic
in the field in actual, close superintendence of constructi

Method of Procedure. Such a surveyor, in taking
architect's or engineer's drawings, readily detects any am
cies, due to hasty preparation or other cause. The attent
engineer is at once called to such matters by the quantit
on with his work. Detected in this way, such uncertaintie
and adjusted, so that by the time the drawings and s
bidders for estimating purposes, everything has been ma
and the possibility of error in quantities can be practic
resulting document, prepared exclusively in the surveyor'
or similarly reproduced, and a facsimile supplied free of c
inserts his unit price opposite each item, and readily fo
in dollars and cents. This is really all that he should
nothing. The quantities of materials and labor then set f
or BILL OF QUANTITIES, should state everything the con
to perform or furnish, in order to complete his contra
becomes a PROPOSAL to do a certain FIXED QUANTITY of
less. This, then, briefly, is the main underlying princ
SYSTEM OF ESTIMATING: a definite quantity of work for a
entire elimination of conditions which now compel bidder
cause all parties, the owners included, to suffer in the end.

The Present Unsatisfactory Conditions. Most a
with the wasteful, unsatisfactory methods followed toda
parties to a contract because of bidders' mistakes in figu
often sacrificed for so-called speed. While wonderful
construction and in materials have been made, no attent
STANDARDIZING METHODS OF ESTIMATING. In the end, of
who suffers. There are, however, as a result of the mov
Francisco, indications that more conservatism and a closer
principles are going to be preferred by the moneyed int
guess-work which has prevailed in the past. Architects
mit an unduly low bidder to take a contract are certainly

Use of the System in Other Countries. The prin
value, or payment by measurement, usually designated as
is based upon equity and square dealing. On large work
Ireland, Scotland, France, Germany, Australia and Sout
significant fact, that in no instance in which this system

ing absorbed in overhead charges, which, of course, are ulti-
e owner.

sputes arising from extra claims which are often made because
f the drawings, and of omissions or other errors in the specifi-

rtunities for the careful, competent bidder. With the quan-
idders all work up from the same basis. The incompetent
it or forget items, and so take work away from more careful
t bidders.

and greater harmony, for the reason that if no part of the
here is less temptation to that letting up on the work which
dissatisfaction, if not in friction or worse.

indings are reduced to a minimum. The bill of quantities is
what is intended, a sort of clearing-house for the drawings and

ty to the contract can obtain any advantage over the other on
ption of work.

with subbidders, it being clearly stated what each trade is

s having much less figuring to do can devote more time and
n to buildings in hand, and especially to supervising and direct-
ictors, a condition much desired at present.

ty of owners and architects to have inferior contractors as the

as, which are usually a trouble to all concerned. Should
ever, they can be more easily adjusted if the schedule prices
ase.

ect or engineer, if he so desires, has the advantage of collab-
professional quantity surveyor, who is available, also, when
s are required. This information is now often furnished by
ius creating an undesirable obligation.

or reorganizing of architects' or engineers' offices is necessary,
ork, now involved in the taking off of figures, would be taken
ntity surveyor's office.

contracts are signed, the drawings and specifications will have
nade as complete as possible, thereby avoiding subsequent
he contractor and his foreman on the job, and largely doing
s at the architect's offices by contractors during the progress

American Practice. In the United States any such system
merican needs and sentiment, and be a practical system. For
ould be undesirable to accept the English practice in either of
reat principles it stands for, however, can be accepted and
advantage.

AND DATA USEFUL IN THE PREPARA- OF ARCHITECTS' DRAWINGS AND SPECIFICATIONS*

r Furniture. For the convenience of draughtsmen when
or providing space for a special article the following dimen-

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litional tables with more detailed and classified lists.
nensions were first contributed to the American Architect of No-
in C. Nye.

Chairs and Seats. The average figures taken from
are: Height of the seat above the floor, 18 in; depth of
of the back above the floor, 38 in. Usually the seat
decreases in height, while the back is higher and slope
inside is a comfortable depth for a seat of moderate size
9 in above the seat. The slope of the back should no
the depth of the seat. A LOUNGE is 6 ft long and abou

Tables vary in shape and size almost as much as cha
dining-tables are made 2 ft 5 in high, and the type of sid
TABLE is made 3 ft high to the principal shelf; but table
6 in high. DINING-TABLES are made from 3 ft 6 in to
from 12 ft to 16 ft by means of slides within the frame.
be so deep as to interfere with the knees of any one sitt
there must be about 2 ft clear space between it and the
practicable for the KNEE-HOLES of desks and library-tabl
wide, the width to be increased as much as possible.

Bedsteads are classed as SINGLE, THREE-QUARTERS,
bed is from 3 to 4 ft wide inside; a three-quarter bed
a double bed, 5 ft. Bedsteads are from 6 ft 6 in to 6 f
boards are from 2 ft 6 in to 3 ft 6 in and headboards fr
Single beds for dormitories are often made only 2 ft 8 i

Bureaus vary in shape and size to such an extent th
to say that any dimension is fixed. Convenient sizes a
1 ft 6 in deep and 2 ft 6 in high; or 4 ft wide, 1 ft 8 in

Commodes are 1 ft 6 in square on the top and 2 ft

Chiffoniers are about 3 ft wide, 1 ft 8 in deep and 4

Cheval-Glasses are made, if large, 6 ft 4 in high and
5 ft high and 1 ft 8 in wide. If medium, 5 ft 6 in high a

Wash-Stands of large sizes are 3 ft long, 1 ft 6 in wide
sizes are from 2 ft 4 in to 2 ft 8 in long.

Wardrobes may be 8 ft high, 2 ft deep and 4 ft 6 in
1 ft 5 in deep and 3 ft wide.

Sideboards may be from 4 to 6 ft long and from 20 i

Upright Pianos vary from 4 ft 10 in to 5 ft 6 in in l
in height and are about 2 ft 4 in deep over all.

Miniature and **Baby-Grand Pianos** vary from 5 ft 10
are about 4 ft 10 in in width.

Parlor-Grand Pianos vary from 5½ ft to 6 ft 10 in i hile some of these measurements vary slightly from the dimen- the preceding paragraphs they represent average dimensions of ade at the present time.

Dimensions of Tables

nd of table	Length	Width	Height	Remarks
le....................	31	22	29	....
le....................	18	18	30	Commode
....	30	22	30	....
....	42	20	36	....
le....................	36	20	30	....
ble...................	66	66	30	Round
ble...................	54	54	30	Square
....	51	41	30	Oval
....	42	27	29	....
....	54	34	29	....
....	60	36	29	....
....	13	13	20	Round
	18	18	24	Square
	23	17	29	Upper shelf
	30	23	18	Lower shelf

s are in inches. Heights are from the floor.

Dimensions of Chairs

chair	Height	Seat-width,		Depth, outside	Back		Arms, height from floor
		Front	Back		Height	Slope	
ir........	18	16	13	17	34	3½	......
chair *....	20	14	12	13½	37	3	27
....	17	29	25	27½	44	4½	......
....	17	22	17½	17	39	.....	......
....	18	22	17	17¾	38	.....	......
....	20	24	22	22	45	2½	26½
....	20	19	17	19	43	2	......
....	19	19	17	18	38½	1½	......
....	18	20	15	15	36	2	......
....	17	33	28	24 ¶	43	5	21
....	17	27	25	27½	41	6½	26
chair.....	18	21½	17	17	34½	2	27
....	16½	24	19½	18¾	36	4	25¾
....	14	21	21	18 ¶	29	.....	......
....	18	26½	22½	26½	37	4	25
....	18	20	13	19	36	3	23
....	20	40	.....	15	......	.....	......
air ‖	17	21	19	21	30	2	......
ir..........	16	23½	20½	19½	41	2	24
chair....	18	18	18	18	29½	0	28½
r..........	20½	17½	17½	15	40	0	......
....	12	18	15	17	28	3	......

2 in above floor. † Overstuffed. ‡ French cane seat and back.
m and back. ‖ *Upholstered seat.* ¶ *Depth inside.*

are in inches. Heights are from the floor. The slope of the back is meas- *level to a perpendicular through the* highest point of the back.

Dimensions of Sofas

Kind of sofa	Height	Seat-width		Depth, outside	Ba
		Front	Back		Height
Small	18	43	40	21	32½
Extra large	16	78	76	36	29
Ordinary sofa	15	54	51	24	34
Lounge	17	68	68	28	35
Lounge	17	57	57	29	23

All dimensions are in inches. Heights are from the floor. The s
measured at the seat-level to a perpendicular through the highest poi

Dimensions of Case-Work

Kind of case-work	Body			
	Width	Depth	Height	
Bureau	45	20½	36½	....
Bureau	51	23	37½	....
Bureau	48	22	36½	....
Bureau	54	20	42	....
Bookkeeper's desk	60	33	42	....
Bookkeeper's desk	60	32	44	Deck,
Chiffonier	39	20	48	....
Chiffonier	36	20	51	....
Cheval-glass	25	....	65	....
Commode	16	16	31	....
Sideboard	84	32	30	....
Wardrobe	36	19	69	....
Wardrobe	54	24	96	....

All dimensions are in inches. Heights are from the floor. The slope
ured at the seat-level to a perpendicular through the highest point of

Dimensions of Bedsteads

Kind of bed	Inside		Heights		s
	Length	Width	Foot	Head	
Single bed	78	42	40	62	
Single bed	78	42	41	60	
Double bed	78	58½	42	63	
Double bed	78	56	36	67	

All dimensions are in inches. Heights are from the floor.

Dimensions of Plumbing-Fixtures. **Enameled-Iron Bath-**
sizes for roll-rim baths with sloping ends are: nominal lengths
5½ ft and 6 ft; width over all, from 30 to 34 in. Specially narr
from 25 to 29 in wide. The actual length over rim is usu
than the nominal length, and 2 in will include an ordinary

s. Crockery basins, to go with marble slabs, are made round
ound bowls are made 10, 12, 13, 14 and 16 in in diam, measured
ide of the rim. Oval bowls, 14 by 17 in, 15 by 19 in and 16 by 21
nd 14-in round, and 15 by 19-in oval, are commonly used.

in-Slabs may be 20 by 24 in, 20 by 30 in, 22 by 28 in, or 24 by 30 in,
a very common size. They can be made any size, to order. They
in thick, countersunk on top, and should have molded edges where

s are commonly made 21 by 21 in and 24 by 24 in. Marble backs
or 10 in high, and sometimes 12 in.

ron Wash-Basins or Lavatories made in one piece: common sizes
n, 11 by 14-in basin; 18 by 21-in, 11 by 15 in basin; 18 by 24 in,
asin; back, 10½ in high. The smallest-sized wash-basin is 13 in
ck.

ns, 12½ by 12½ in, 12-in round basin; 15 by 15 in, 11 by 14-in
6 in, 11 by 14-in basin; 19 by 19 in, 11 by 15-in basin. The stand-
wash-basins is 2 ft 6 in from the floor.

enameled iron, roll-rim, are 22½ by 19 in; width, including
1 in; height 17 in; depth inside, 11 in.

enameled iron, average about 32 in long over fittings, and 27 in

ets. The dimensions of water-closet bowls vary considerably,
being about an average: width of bowl over all, 13 in; depth from
of seat, 23 in; height from floor to seat, 17 in; width of seat, from
Closets with low-down tanks measure about 28 in from front of
The distance from center of outlet-opening to the walls, or the
dimensions, are given in manufacturers' catalogues, as they vary
closets. The smallest space permissible for water-closet compart-
doors open out, is 2 ft 4 in by 4 ft. If the doors open in, the com-
uld be 3 by 5 ft.

es, used in schools and factories, are made 24, 27 and 30 in,
ter of partitions. For graded schools, 24 in is ample, and for
. The range usually occupies a space 28 in in depth, if set against

s should be from 24 to 27 in, center to center of partitions; depth
20 or 22 in; of ends, 2 ft; of bottom slab, 2 ft; height of partitions,
to 5 ft 6 in.

ks of cast iron are made in a great variety of sizes, those most
d being 16 by 24 in, 18 by 30 in, 18 by 36 in, 20 by 30 in and 20 by
in is the largest size for enameled sinks. The depth inside, for
, is 6 in. Plain cast-iron sinks are made as large as 32 by 56 in, or
Steel sinks are made in all of the above sizes up to 20 by 40 in.

nks. Common sizes of porcelain sinks are 20 by 30 in, 23 by 36 in
in.

lop-Sinks, common sizes, are 16 by 16 in, 16 by 20 in, 18 by 22 in
in; 12 in deep.

try-Sinks. Common sizes are 12 by 18 in, 14 by 20 in and 16 by

of slate or soapstone are commonly made 2 ft wide over all,
Lengths over all, two-part tubs, 4 ft and 4 ft 6 in; three-

Landau. Length over all, 19 ft 6 in; width, 6 ft 3 in; he
of pole, 8 ft 0 in.

Stanhope Gig, Two Wheels. Length over all, 10 ft 6 i
height, 7 ft 6 in.

Victoria. Length, without pole, 9 ft 6 in; length of p
all, 5 ft 4 in.

Light Brougham. Length, without pole or shaft, 9 to
5 ft 4 in; height, 6 ft 4 in.

Automobiles. Length, from 11 to 19 (average 16) ft; wi

Dimensions and Weight of Fire-Engines. From m
ent fire-engines belonging to the city of Boston, it was fo
length, including pole, was 22 ft 6 in. The widths varied
the average height being 8 ft 8 in. The average weigh
engines), 8 000 lb; the greatest weight, 9 420 lb and the l

Dimensions and Weight of Hose-Carriages. Extre
19 ft 6 in, without horse, 17 ft 6 in; width, from 5 ft 9 in to
8 in to 7 ft; average weight (computed from 11 carriage
weight, 3 500; least weight, 2 120.

Dimensions and Weight of Ladder-Wagons. Len
total length, with ladders on, 45 ft; width, 6 ft 2 in; a
puted from 12 wagons), 6 660 lb; greatest weight, 8 800;

Dimensions of Locomotives and Cars. The dime
and freight-cars vary considerably, but the following will
mon use:

Locomotives. From 15 ft 4 in to 15 ft 10 in to top of s
extreme width of cab, 10 ft 2 in. Doors to admit locom
12 to 13 ft wide and 18 ft high.

Furniture-Cars are 14 ft 1 in, from top of track to top ches vary from 14 to 16 ft in height and from 10 to 11 ft in o admit cars should give at least 12 in clearance on each side, d.

Cars are about 8 ft 6 in wide for the car proper, and the steps n. Height from track to top of coach, 11 ft 6 in; the trolley- gher. The length varies, up to 42 ft. Trucks for a 41 ft 6 in ft apart. Wheel-bases, 4 ft center to center. Radius of short- ver, Colo., 35 ft to midway between rails. The GAUGE of a rail- distance between the inner sides of the heads of the two rails. BROAD GAUGE is 4 ft 8½ in; standard NARROW GAUGE, 3 ft 3½ in.

reight-Cars. Car-Loads. The capacity of freight-cars, and r-loads, vary so greatly that no accurate general information 'or heavy freight, 25 tons is an average load; for light freight, ns; for household goods, 10 tons is about the minimum; for about a minimum load; for cement, 20 tons. The minimum in car-load rates, varies with different roads, and also with the v rate is usually made on the basis of a big load. Thirty tons r heavy freight, and 40 tons is about the maximum, except for

s Dimensions. Horse-Stalls. Width, from 3 ft 10 in to 4 ft ver; length, 9 ft. The width should never be between 4 ft and liable to cast himself.

Standard Bowling-Alleys.* For ONE PAIR OF ALLEYS: Room over all; 11 ft 6 in wide, 60 ft from foul-line to head pin, 3 ft of alley, 4 ft for pin-pit, 8 in deep in front, 6 in in back; alleys, , should extend on and beyond the foul-line 12 ft, and then a 16-ft approach to the foul-line for the player to run to deliver E ALLEY: Same length, 83 ft; width, 6 ft 3¼ in; closer dimen- , gutters 9 in, division-pieces 2¾ in, ball-return 9¾ in.

	In		In
ll-return	9¾	ONE PAIR OF ALLEYS: Ball-return	9¾
ce	2¾	First-division piece	2¾
	9	Gutter	9
	42	Bed	42
	9	Gutter	9
piece	2¾	Second-division piece	2¾
6 ft 3¼ in =	75¼	6 ft 3¼ in =	75¼

of the PAIR OF ALLEYS, should be added

	9
	42
	9
iece	2¾
	138

m should be provided for the bowlers and spectators as these or the alleys only.

Drawings for Patents *(United States).* *10 by 15 in, with nside all around.*

hed by The Brunswick-Balke-Collender Company, New York City.

abandoned the use of wooden flagpoles and is using stee
ordinary building, 60 ft in height above the curb, a pole 43
which is sufficient for the tackle of a large or post-flag, for
parapets are very low. Each pole is required to be fitted
iron, galvanized, revolving truck, mounted on crucible-st
neath it, also, being of galvanized iron. The truck is fit
bronze sheaves on Tobin-bronze pins, surmounted with
ball, acid-cleaned and painted with four coats of the best E
sizing, and covered with XXXX leaf-gold. One or more
mitted in the length of the pole, which are determined a
details, the bands being secured to the male tube, and bot
band and the shoe being machine-beveled to insure a per
tube is drilled and secured to the male shoe with tap-screws
to carry the upper section of the pole, and the ends of t
The exposed ends of the female tube are chamfered and ca
collar or band, to receive the copper flashing, is secured to
just above the roof-lines.

Dimensions of Schoolrooms, Boston Schools.‡ The
the Boston school-houses, as adopted by the school boar
schools, 28 by 32 ft in plan by 13 ft 6 in in height; for pr
32 by 12 ft. This accommodates 56 scholars per room, in
216 cu ft per scholar in the grammar schools, and 165 cu ft i
A width of 27 ft is very satisfactory for schoolrooms, and i
because it permits of the use of 28-in joists, without waste.

Heights of Blackboards in Schoolrooms.‡ The heig
of chalk-rail should be about as follows:

For third and fourth grades,	chalk-rail.	2 ft 1
For fifth grade,	chalk-rail.	2 ft 2

Sizes of Seats and Desks for Schools and Academies *

r of	Age of scholar years	Height of seat or chair in	Height of desk (next scholar) in	Space occupied by desk and seat (back to back) ft	 in
	16 to 18	16¾	29½	2	9
	14 to 16	15½	28	2	9
	12 to 14	15½	27½	2	8
	10 to 12	14¼	26½	2	7
	8 to 10	13¼	25½	2	5
	7 to 8	12¼	24	2	4
	6 to 7	11½	22½	2	3
	5 to 6	10½	21	2	2
.....	4 to 5	9½	19	2	0

two scholars are 3 ft 10 in long, and for a single scholar, 2 ft long.
from 2 ft to 2 ft 4 in wide, according to age of scholars and size of room.

Additional Data† on School-Houses

Rooms. The Department of Education, New York City, has adopted, nensions of the schoolrooms, the German standard of 22 by 30 ft in ft in height, with unilateral lighting. These dimensions are used for of elementary schools, the sittings being on the basis of 15 sq ft of per pupil. Good light cannot be had on desks which are placed at a tance from the windows than one and one-half times the height of the ipper sash from the floor.

izes of Seats and Desks for Elementary and High Schools

of	Age of scholar years	Height‡ of seat or chair in	Height‡ of desk in	Space§ occupied by desk and seat (back to back) in
	16 to 18	17	31	32
	14 to 16	16	30	32
	12 to 14	15	28	31
	10 to 12	14	26	30
	8 to 10	13	24	29½
	7 to 8	12	23	27
	6 to 7	11	22	27
	5 to 6	10	20½	26

rds. For first and second-year scholars the chalk-rail is placed 2 ft oor, and the boards are 4 ft high. This allows the smaller children

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idder, in previous editions.

ata compiled by E. S. Hand from notes furnished by C. B. J. Snyder, Super- School Buildings, New York City.

are measured as follows: From the floor to the top of ink-well strips of rom floor to top of front edge of seats, and should not vary more than ½ in ghts given in this table.

re a minimum width of 18 in for the lower grades and 22 in for the upper

re used, this distance must be increased from 1½ to 2 in.

Table of Treads and Risers *

The figures in the first column can be used for either treads or risers

Height of riser or width of tread, inches	Number of treads or risers																									
	2		3		4		5		6		7		8		9		10		11		12		13		14	
	Story-height or horizontal length of run																									
	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in
6	1	0	1	6	2	0	2	6	3	0	3	6	4	0	4	6	5	0	5	6	6	0	6	6	7	0
6¼	1	0½	1	6¾	2	1	2	7¼	3	1½	3	7¾	4	2	4	8¼	5	2½	5	8¾	6	3	6	9¾	7	3½
6½	1	1	1	7½	2	2	2	8½	3	3	3	9½	4	4	4	10½	5	5	5	11½	6	6	7	0½	7	7
6¾	1	1½	1	8¼	2	3	2	9¾	3	4½	3	11¼	4	6	5	0¾	5	7½	6	2¼	6	9	7	3¾	7	10½
7	1	2	1	9	2	4	2	11	3	6	4	1	4	8	5	3	5	10	6	5	7	0	7	7	8	2
7⅛	1	2¼	1	9⅜	2	4½	2	11⅝	3	6¾	4	1⅞	4	9	5	4⅛	5	11¼	6	6⅜	7	1½	7	8⅝	8	3¾
7¼	1	2½	1	9¾	2	5	3	0¼	3	7½	4	2¾	4	10	5	5¼	6	0½	6	7¾	7	3	7	10¼	8	5½
7⅜	1	2¾	1	10⅛	2	5½	3	0⅞	3	8¼	4	3⅝	4	11	5	6⅜	6	1¾	6	9⅛	7	4½	7	11⅞	8	7¼
7½	1	3	1	10½	2	6	3	1½	3	9	4	4½	5	0	5	7½	6	3	6	10½	7	6	8	1½	8	9
7⅝	1	3¼	1	10⅞	2	6½	3	2⅛	3	9¾	4	5⅜	5	1	5	8⅝	6	4¼	6	11⅞	7	7½	8	3⅛	8	10¾
7¾	1	3½	1	11¼	2	7	3	2¾	3	10½	4	6¼	5	2	5	9¾	6	5½	7	1¼	7	9	8	4¾	9	0½
7⅞	1	3¾	1	11⅝	2	7½	3	3⅜	3	11¼	4	7⅛	5	3	5	10⅞	6	6¾	7	2⅝	7	10½	8	6⅜	9	2¼
8	1	4	2	0	2	8	3	4	4	0	4	8	5	4	6	0	6	8	7	4	8	0	8	8	9	4
8¼	1	4½	2	0¾	2	9	3	5¼	4	1½	4	9¾	5	6	6	2¼	6	10½	7	6¾	8	3	8	11¼	9	7½
8½	1	5	2	1½	2	10	3	6½	4	3	4	11½	5	8	6	4½	7	1	7	9½	8	6	9	2½	9	11
9	1	6	2	3	3	0	3	9	4	6	5	3	6	0	6	9	7	6	8	3	9	0	9	9	10	6
9½	1	7	2	4½	3	2	3	11½	4	9	5	6½	6	4	7	1½	7	11	8	8½	9	6	10	3½	11	1
10	1	8	2	6	3	4	4	2	5	0	5	10	6	8	7	6	8	4	9	2	10	0	10	10	11	8
10½	1	9	2	7½	3	6	4	4½	5	3	6	1½	7	0	7	10½	8	9	9	7½	10	6	11	4½	12	3
11	1	10	2	9	3	8	4	7	5	6	6	5	7	4	8	3	9	2	10	1	11	0	11	11	12	10
[illegible]	2	2	3	3	4	4	5	5	6	6	7	7	8	8	9	9	10	10	11	11	13	0	14	1	15	2
[illegible]	2	4	3	6	4	8	5	10	7	0	8	2	9	4	10	6	11	8	12	10	14	0	15	2	16	4

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* [illegible]

Tread, inches	Story-height or horizontal length of run																									
	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in
6	7	6	8	0	8	6	9	0	9	6	10	0	10	6	11	0	11	6	12	0	12	6	13	0	13	6
6¼	7	9¾	8	4	8	10¼	9	4½	9	10¾	10	5	10	11¼	11	5½	11	11¾	12	6	13	0¼	13	6½	14	0¾
6½	8	1½	8	8	9	2½	9	9	10	3½	10	10	11	4½	11	11	12	5½	13	0	13	6½	14	1	14	7½
6¾	8	5¼	9	0	9	6¾	10	1½	10	8¼	11	3	11	9¾	12	4½	12	11¼	13	6	14	0¾	14	7½	15	2¼
7	8	9	9	4	9	11	10	6	11	1	11	8	12	3	12	10	13	5	14	0	14	7	15	2	15	9
7⅛	8	10⅞	9	6	10	1⅛	10	8¼	11	3⅜	11	10½	12	5⅝	13	0¾	13	7⅞	14	3	14	10⅛	15	5¼	16	0⅜
7¼	9	0¾	9	8	10	3¼	10	10½	11	5¾	12	1	12	8¼	13	3½	13	10¾	14	6	15	1¼	15	8½	16	3¾
7⅜	9	2⅝	9	10	10	5⅜	11	0¾	11	8⅛	12	3½	12	10⅞	13	6¼	14	1⅝	14	9	15	4⅜	15	11¾	16	7⅛
7½	9	4½	10	0	10	7½	11	3	11	10½	12	6	13	1½	13	9	14	4½	15	0	15	7½	16	3	16	10½
7⅝	9	6⅜	10	2	10	9⅝	11	5¼	12	0⅞	12	8½	13	4⅛	13	11¾	14	7⅜	15	3	15	10⅝	16	6¼	17	1⅞
7¾	9	8¼	10	4	10	11¾	11	7½	12	3¼	12	11	13	6¾	14	2½	14	10¼	15	6	16	1¾	16	9½	17	5¼
7⅞	9	10⅛	10	6	11	1⅞	11	9¾	12	5⅝	13	1½	13	9⅜	14	5¼	15	1⅛	15	9	16	4⅞	17	0¾	17	8⅝
8	10	0	10	8	11	4	12	0	12	8	13	4	14	0	14	8	15	4	16	0	16	8	17	4	18	0
8¼	10	3¾	11	0	11	8¼	12	4½	13	0¾	13	9	14	5¼	15	1½	15	9¾	16	6	17	2¼	17	10½	18	6¾
8½	10	7½	11	4	12	0½	12	9	13	5½	14	2	14	10½	15	7	16	3½	17	0	17	8½	18	5	19	1½
9	11	3	12	0	12	9	13	6	14	3	15	0	15	9	16	6	17	3	18	0	18	9	19	6	20	3
9½	11	10½	12	8	13	5½	14	3	15	0½	15	10	16	7½	17	5	18	2½	19	0	19	9½	20	7	21	4½
10	12	6	13	4	14	2	15	0	15	10	16	8	17	6	18	4	19	2	20	0	20	10	21	8	22	6
10½	13	1½	14	0	14	10½	15	9	16	7½	17	6	18	4½	19	3	20	1½	21	0	21	10½	22	9	23	7½
11	13	9	14	8	15	7	16	6	17	5	18	4	19	3	20	2	21	1	22	0	22	11	23	10	24	9
13	16	3	17	4	18	5	19	6	20	7	21	8	22	9	23	10	24	11	26	0	27	1	28	2	29	3
14	17	6	18	8	19	10	21	0	22	2	23	4	24	6	25	8	26	10	28	0	29	2	30	4	31	6

* The editor is indebted to T. Z. Talley for the calculations and arrangement of this table.

to use the lower portion. The upper part of the surface is a
for the use of the teacher, there being much display-work e
grades. For scholars in grades from the third to the eight
for high schools the chalk-rail is placed 2½ ft from the floo
3 ft 6 in in height.

Doors and Stairways. Wardrobes should be entered
only. Classroom-doors should open into the rooms, so as
control in case of panic. All exit-doors should open out.
be shut off from corridors by means of self-closing doors,
the stairways and the enclosures, should be of fire-proof m
should be of sufficient number to permit of the building
three minutes from the time a signal is given. This can be
a linear width of 4 ft for the first 50 persons and 12 in add
persons in excess thereof. No stairway is to be less than
5 ft in width. Exits should be planned so as to provide 15
persons and 6 in additional for each 100 persons in excess th
should have more than 15 steps in any one flight, change
effected by a square platform and no winders being us
exit-door should open out over a step. Platforms are to b
doors and are to extend at least 1 ft beyond the edge of the
open.

Stairs.* The RISE of a stair is the height from the top o
of the next. The TOTAL RISE is the height from floor to flo
horizontal distance from the face of one riser to the face o
are the upright boards or other materials forming the fac
the TREADS are the horizontal pieces or surfaces on wh
Treads are usually from 1¼ to 1¾ in wider than the ru
NOSING. The height of an individual riser or the RISE of
by dividing the TOTAL RISE by the number of risers. Th
may be fixed at will unless the space is cramped, but to s
stair the run must bear a certain relation to the rise.

Rules for Dimensions of Treads and Risers. For ordina
7 to 7½ in makes a very comfortable flight of stairs.
stairs used by children the rise should not exceed 6 in.
greater than 7¾ in are steep. The width of the run should
the height of the rise; the less the rise the greater should b
versa. Several rules have been given for proportioning the

(1) THE SUM OF THE RISE AND RUN should be equal to fro

(2) THE SUM OF TWO RISERS AND A TREAD should not be le
than 25 in.

(3) THE PRODUCT OF THE RISE AND RUN should not be le
than 75.

These rules apply only to stairs with nosings. Stone st
should have at least 12-in treads for adults. (See Tables,

ds.* Until a few years ago, linen or cotton cord only was used for weights with the sashes of double-hung windows, and cord is still ively used than either ribbons or chains. For windows of ordinary braid of cord will wear for a long time, and this material will probe entirely displaced by metal. "Tests made at the Massachusetts Technology show that cords wear much longer than chains, though ss tensile strength. Cords should be smooth and round, so that each its part of the stress, and well glazed, so that they have a smooth consequently less wear from friction with the wheel of the pulley." found that cord can be braided too hard for durability, yet if it is s to be very flexible it may be so soft that it will stretch and cause ance by permitting the weight to hit the bottom of the weight-box. t, however, should always specify the particular BRAND and SIZE e used, and also the diameter of the pulley. Among the leading sh-cord at present are the Samson Spot,† and the Silver Lake A.‡ s are superior to the ordinary braided cords, which are made from s to meet the jobbers' requirements for price. In addition to other nt qualities, the Samson cord offers an additional advantage that ll appreciate; it has a colored strand woven through it, which shows he surface and thus enables one to tell at a glance that no other cord stituted. The Silver Lake A sash-cord has the name Silver Lake A every foot of cord; but unless the letter A accompanies the name a e of cord is denoted. The marking of the cord by color, or any , does not alter the quality of the cord. Special marks may be ferior cords as well as to the best. The following numbers should for the different weights of sash-weights:

Relative Sizes of Sash-Cords, Weights and Pulleys

er	6	7	8	9	10	12
n inches	3/16	7/32	1/4	9/32	5/16	3/8
und	66	55	44	36	27	20
for weights in p to	5	12	20	30	40	50
liameter in inches allowable	1½	1¾	2	2¼	2½	3

ing sashes weighing over 40 lb, only the largest size of Samson or A cord, or some form of sash-chain or sash-ribbon, should be used, eys should be selected to fit the cord or chain. A guarantee that the t at least twenty years may be had from either of the manufacturers bove. The Samson wire-center sash-cord has recently been put on This is really a metal sash-cord protected by a braided-cotton surcts as a noiseless cushion. It is claimed that it harmonizes with the h and that it has greater durability than other sash-cords or metal ee record of tests made at Massachusetts Institute of Technology, The standard color is that of dark mahogany, but this cord is er for large buildings in other colors to match the finish.

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wing notes, relating to Sash-Cords, Sash-Chains, Sash-Ribbons, Sash-ash-Balances, are condensed and revised from articles by Professor Thomas dder's Building Construction and Superintendence, Part II, Carpenters'

red by the Samson Cordage Works, Boston, Mass.
ed by the Silver Lake Company, Boston, Mass.

Sash-Chains. Of several styles of sash-chains on the market, the styl
largely used is the flat-link chain.* This chain is made either of steel, or of
composed of 95% copper and 5% of tin. For suspending very heavy
doors and gates, a cable-chain has been extensively used. Star † sash-c
made of bronze-metal. The manufacturers of the Norris sash-pulley chai
a riveted chain that has joints only one way is almost sure to break whe
slightly twisted, and that it is better to use two chains of the link-patter
ning side by side over the same pulley. The strongest sash-chains are of
made rust-proof by the hot-galvanizing process, and electro-copperplat
give a bronze finish; and of a bronze-mixture which looks like copper,
tougher and harder. One firm ‡ claims that its galvanized-steel sash-c
from 11 to 45% stronger than any bronze or copper sash-chain and that
resist fire for a much longer period. The tensile strength of their chain
from 475 to 850 lb, according to the weight used.

Sash-Ribbons. These are now also extensively used in hanging the sa
the better class of buildings. The ribbons are made of steel and alum
bronze or of some mixture of aluminum, and in ⅜, ½, ⅝, ¾ and ⅞-in
They are claimed to be practically indestructible, but according to one s
tests it would appear that in some cases they do not wear as long as sas
or sash-chains. Some people object that the ribbons snap against the
stiles, when the sash is raised or lowered, and thus make considerabl
The ⅜-in ribbon may be used for a sash weighing up to 100 lb and rn
50-lb weights. For a window 6 ft 10 in high and 3 ft wide, glazed with plat
the ribbons with attachments cost about 75 cts. Sash-ribbons are now
factured by a number of firms who also make the necessary attachme
weight and sash. For the best working of windows hung with ribbons,
of the following sizes should be used:

For sashes weighing not over 40 lb, 2 in
For sashes weighing not over 60 lb, 2¼ in
For sashes weighing not over 100 lb, 2½ in
For sashes weighing not over 150 lb, 3 in
For sashes weighing not over 250 lb, 3½ in
For sashes weighing not over 300 lb, 4 in
For sashes weighing not over 350 lb, 4½ in

Comparative Strength of Sash-Cords and Chains. The comparative
and durability of sash-cords and chains have been determined by caref
but there is a great variation in both cases, due partly to variation i
rial, but principally to the relative sizes of the chain and pulley or
pulley. The cords or chains may be too light for the weights used
pulleys too small in diameter to carry the cord without undue bendin
pulleys may also have too narrow a groove or an uneven groove wit
edges which cut the cords. The larger the diameter of the pulley, th
the wear.

Tests § on Wire-Center Sash-Cord and Bronze Sash-Chains.
tested was size No. 8, ¼-in diam, Samson solid braided cotton cord

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* One type of this kind of sash-chain is manufactured by the Bridgeport
pany, Bridgeport, Conn.

† Manufactured by the U. T. Hungerford Brass & Copper Company, New Y

‡ The Oneida Community, Ltd., Oneida, N. Y.

§ Made at the Massachusetts Institute of Technology, May, 1914, by
Miller.

center, 5⁄16 in in diam. The chains tested were of two different
ronze, size No. 2, purchased in the open market as typical bronze
each recommended by a reputable dealer as the proper chain for
25-lb window-weight. The tests for the better of the two chains are
. Durability-tests were made by raising and lowering a 25-lb
a 2-in pulley, each movement corresponding to once opening and
window. The cord was tested over the regular round grooved
arily used for cords, and the chains were tested over the combina-
d pulley usually furnished for sash-chains. For the fire-tests the
ains were hung through an asbestos box in which a Bunsen flame
ure was applied to all alike, the temperature being about 2 200° F.
ght was attached in each case to keep the cord or chain under the
n. The wire-center cord took about twice as long to burn through
bout seventeen times as long as the bronze chain.

ests on Wire-Center Sash-Cord and Bronze Sash-Chain

Durability-tests		Fire-tests	
er of lifts before breaking		Length of time before parting	
chain	Samson wire-center cord	Bronze chain, sec	Samson wire-center cord, sec
944	659 892	42.5	78.5
486	592 559	40	75.5
381	632 230	39	77
948	594 114	32	75
356	631 286	....	....
234	577 154	....	....
790	504 032	....	....
874	637 796	....	....
377	Average 603 633	Average 38.4	Average 76.5

of Sashes and Glass. In figuring the weights of windows, the
he glass may be taken at 3½ lb per sq ft for plate glass, 1⅓ lb for
ngth glass and 1 lb for single-strength glass. For the weight of the
h, add together the height and width, in feet, of each sash, and mul-
for 2¼-in sash, by 1⅔ for 1¾-in sash and by 1⅓ for 1⅜-in sash.
s are sufficiently accurate for determining the size of sash-cords
, but the weights should be determined by weighing each sash after
as the weight of the glass varies considerably.

h-Weights. The weights ordinarily used for balancing windows are
t iron, in the form of solid cylinders from 1⅜ to 2⅛ in in diameter, and
31 in long, with an eye cast in the upper end of each. The lengths
he weights, which are from 2 to 25 lb. Flat weights, which usually
r in the Philadelphia and some other markets, are from 6 to 34½ in
2 to 30 lb in weight, and from 1¼ by 1⅝ to 1⅝ by 2¼ in in cross-
ordering sash-weights the *number of pounds of each weight, and*
and lengths of the boxes in which the weights will work, should be
ary weights have very rough eyes for the sash-cords. There are
turers in the East who make weights with a patent eye that will

not cut the cord. A sectional sash-weight[*] made with a well-desig
device which has given satisfaction, is said to be one of the best o
Usually from three to six sections are required on each side to b
properly. If the hooking-device fails near the top the upper sa
closed and if at the bottom the window cannot be opened. It is t
to open the weight-box and rehang the sections before the window
ated. In theory, sectional weights are ideal; in practice, howe
not considered as satisfactory as solid weights. The Brown[†] sect
are made 2¼ by 2⅜ in and in weights of 6, 7, 8, 9 and 10 lb. T
of both cast-iron and lead. It frequently occurs after a contract i
glass is changed from double-thick to plate or prism glass. This m
weight; but the length of the sash-weight cannot be increased and
becomes necessary to increase the area of its cross-section. If t
is detailed to take the regular round sash-weight, its general con
be such that it will take a 2-in round sash-weight, but not a 2-in
weight. This difficulty can be avoided by a little thought at t
added depth of ¼ in in the weight-box permits the use of a rectan
sash-weight. The Sanborn sectional sash-weight[‡] is intended f
buildings of heavy construction. Because of the lack of unif
weight of plate glass the required weights of sash-weights cannot
determined previous to the hanging of the sashes. By the use
sash-weight, combinations of units can be made up to suit the
The units are made square or rectangular in section in order to
imum weight with a minimum length. An opening of 12 in in t
pocket is sufficient for hanging the largest unit. These units are
in standard sizes to meet the general conditions found in the bui

Lead Sash-Weights. It often happens that for wide and lo
weights if of iron would be so long that they would touch the
pocket before the bottom sash was fully raised. In such cases le
usually resorted to, lead being 80% heavier than cast iron.
weights square in section, whether of iron or lead, a considerable
made in the lengths. One sash-weight manufacturer[§] makes
compressed-lead sash-weights, with wrought and malleable-i
centered so that the weights hang perfectly plumb; and when le
necessary the architect will do well to specify the weights mad
pany. These weights are made under hydraulic pressure, by
smoothness, solidity and density of metal is secured than is
casting-process. A wrought-iron rod is run through the cente
securely attached the malleable-iron fittings. In hanging the sas
for the upper sash should be about ½ lb heavier than the sash, a
sash, ½ lb lighter.

Sash-Balances. Within a comparatively few years severa
been patented for balancing sashes by means of springs instead
the author believes that only one type, known as the SASH-BALA
a practical success. The sash-balance consists of a drum on w
is wound, and which contains a coiled-steel clock-spring, imme
spring sustains the weight of the sash. The common type very
in outward appearance the ordinary sash-pulley, and is applic
the same way; the ribbons, which are made usually of alumi

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[*] Manufactured by E. E. Brown & Company, Philadelphia, Pa.

[†] Manufactured by E. E. Brown & Company, Philadelphia, Pa.

[‡] Manufactured by the Lidgerwood Manufacturing Company, N

[§] Raymond Lead Company, Chicago, Ill.

the sashes in the same manner as cords when weights are used.
sh-balance in its best form works very satisfactorily, it will probably
y supplant the weight and axle-pulley for ordinary windows. There
ndows, however, for which sufficient pocket-room for weights cannot
without spoiling the effect desired or narrowing the glass, as in some
s, or where it is undesirable to break the frame into the brick jamb.
es the sash-balance is almost invaluable. For hanging the glass
ow-cases, sash-balances are usually preferable to weights. Sash-
made in both side and top-patterns, but the former are recommended
ere is room at the side of the frame for the depth of mortise required.
s of the sizes usually found in residences, the depth of the sash-
sured from the face of the pulley-stile will vary from 3 to 4 in;
provided for usually by cutting a hole, if necessary, in the masonry
back of the frame. As sash-balances require only a plank frame,
ent reduction in the cost of the frame offsets the extra cost of the
remodeling old buildings which have plank frames without weights,
s are found to be a great convenience, since they can easily be in-
e old frames. An advantage which all spring-balances possess is
t most strongly when the sash is down, and so enable one to raise
indow more readily than if it were hung with weights; while when
up the springs barely suffice to hold it in position, and do not offer
drawing it down. Of the various sash-balances on the market, the
d the Caldwell † are the most extensively used, and are undoubtedly
he Pullman Unit sash-balance has been on the market many years
ved satisfactory. These balances are now made with uniform-size
or the various weights of sash with which they are to be used, and
t possible to have all mortises for the balances cut at the mill, as is
or the regular cord-pulleys. The Caldwell sash-balance, both top
es, is much used by the United States Post Office and Navy Depart-
s used also by the leading car-builders. The springs are made of
old-rolled tempered-steel wire, a material similar to that used for
s. The manufacturers guarantee these sash-balances for from ten
ars.

Space in Churches and Theaters. The minimum spacing for
to back, is 30 in. This spacing is fairly comfortable for occupants,
le cramped for persons passing by others into or out of the pews.
32 in is to be preferred, and if there is abundance of room, the spac-
made 33 in. Anything over 33 in is a waste of room. A space of
length of the pew is considered a SITTING.‡

Theater-Chairs are made 19, 20, 21 and 22 in wide, center to center
d in arranging them in rows where the aisles converge, the ends are
line on the aisles by using a few chairs that are either narrower or
the standard width. For churches, a standard width of 20 in is
at is desirable. For theaters, 21 or 22-in chairs are commonly used
uet, 20 or 21-in in the dress-circle, and 20 and 19-in in balcony and
ough there is no accepted rule in this respect. On account of the
opera or theater-chairs may be comfortably spaced 31 in, back to
his is the usual spacing in halls and churches. In theaters the
sually set on steps. In the upper gallery these steps should not be
o in wide; in the balcony they are usually made either 30 or 31 in

tured by the Pullman Manufacturing Company, Rochester, N. Y.
ured by the Caldwell Manufacturing Company, Rochester, N. Y.
ions of pew-bodies see page 48 of Churches and Chapels, by F. E. Kidder.

wide, and in the parquet, 31 or 32 in wide. As a rule the higher-
are more commodious than the lower-priced.

Estimating Seating Capacity. The actual seating capacity of t
audience-rooms can be determined only by drawing the seats to
scale, on the floor-plan, and then counting the number of chairs,
the linear feet of pews.

Approximate Seating Capacity. For approximate purposes the s
ity or required size of room may be determined by allowing fro
to each seat, or sitting, when on a curve, and from 6 to 7 sq ft t
when in straight rows, the smaller number being used only for
This allows for aisles and pulpit-platform. For small concert-hal
rectangular rooms, 6 sq ft per sitting will usually be sufficient al
vided only that the actual floor-space utilized for seats and aisles

Seating Capacity of Several of the Older Cathedrals, Churches and Opera-Houses *

European Cathedrals and Churches Estimating that one person occupies an area of 19.7 square i		
St. Peter's, Rome............	54 000	Notre Dame, Paris......
Milan Cathedral.............	37 000	Pisa Cathedral..........
St. Paul's, Rome............	32 000	St. Stephen's, Vienna...
St. Paul's, London..........	25 600	St. Dominic's, Bologna..
St. Petronio's, Bologna.....	24 400	St. Peter's, Bologna.....
Florence Cathedral..........	24 300	Cathedral of Sienna.....
Antwerp Cathedral..........	24 000	St. Mark's, Venice......
St. Sophia's, Constantinople.	23 000	Spurgeon's Tabernacle, London..............
St. John Lateran, Rome....	22 900	
European Theaters and Opera-Houses		
Carlo Felice, Genoa.........	2 560	Drury Lane, London....
Opera-House, Munich.......	2 370	Covent Garden, London
Alexander, St. Petersburg...	2 332	Opera House, Berlin.....
San Carlo, Naples...........	2 240	Adelphi, London........
Imperial, St. Petersburg.....	2 160	Lancaster, London......
La Scala, Milan.............	2 113	Globe, London..........
Academy of Paris...........	2 092	....
Some Early American Theaters and Opera-Houses, outside of N		
The Auditorium, Chicago...	4 200	Castle Square Theater, Boston................
Academy of Music, Philadelphia....................	3 124	Gaiety Theater, Boston..
Boston Theater, Boston.....	3 000	Grand Opera-House, C cinnati...............
....	....	

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* The table following this one gives the seating capacities of theaters i
some of the boroughs of New York City. The above table of seating ca
of the earlier churches and theaters is retained for purposes of compa
important structures of these types have been erected in recent years in t
the world, or are now (1915) in process of erection, that it has been
make any list that would be and would remain, for any length of t

Seating Capacity of New York Theaters (1914)

Boroughs of Manhattan and the Bronx			
ame	Seating capacity	Name	Seating capacity
Music	2 653	Gaiety	806
	1 389	Garden	1 090
	1 683	Garrick	844
oof	1 134	Globe	1 194
	1 137	Gotham	1 626
	984	Grand	1 888
um	416	Grand Opera-House	2 093
	814	Greeley Square (Loew's)	1 906
	1 776	Harlem Casino	*
	1 764	Harlem Opera-House	1 393
ll	2 632	Harris	847
eum	640	Herald Square	1 160
	1 465	Hippodrome	4 588
	2 078	Hudson	1 077
of	1 150	Hurtig and Seamon's (Music-Hall)	1 093
	1 684		
	2 289	Illington	*
et (Odeon)	904	Irving Place	1 079
ohan	1 072	Keith's Union Square	1 080
	1 541	Kessler's 2nd Avenue	1 803
	1 315	Kessler's 2nd Avenue, Roof	734
	696	Knickerbocker	1 351
	916	Lafayette	1 042
	1 074	Liberty	1 211
et	1 242	Lincoln Square	1 547
y)	1 310	Lipzin	1 030
	1 436	Little	299
	895	Longacre	*
	1 099	Loew's Fifth Avenue	964
	687	Loew's 7th Avenue	1 626
e (Proctor's)	1 204	Loew's National	2 333
	1 255	Lyceum	957
Brady's)	969	Lyric	1 452
	662	Madison Square Garden	3 366

* Data not furnished.

Seating Capacity of New York Theaters (1914) (Continued

Boroughs of Manhattan and the Bronx

Name	Seating capacity	Name
Madison Sq Garden, Roof	700	Proctor's 23rd Street
Manhattan Opera-House	3 200	Proctor's 58th Street
Maxine Elliott	904	Proctor's 125th Street
Metropolis	1 150	Prospect
Metropolitan Opera-House	3 305	Republic
McKinley Square	1 500	Richmond
Miner's Bowery	1 168	Riverside
Miner's Bronx (Acme)	1 798	Savoy
Miner's 8th Avenue	1 178	Star
Minsky	1 866	St. Nicholas
Moulin Rouge	1 615	Thalia
Murray Hill	1 224	Third Avenue (Keeney's)
Mount Morris	*	39th Street
Nemo	941	Tremont
New Amsterdam	1 618	Victoria
New Amsterdam, Roof	610	Victoria, Roof
New York Theater, Roof	1 337	Wadsworth
Olympic	745	Wallack's
116th Street	1 743	Washington
Odeon 145th Street	904	Weber's
Park	1 513	West End
People's	1 693	Weber and Field Music-Hall
Philipps	*	Winter Garden
Plaza	1 544	Yorkville
Playhouse	879	

* Data not furnished.

Borough of Brooklyn

Name	Seating capacity	Name
Academy of Music	2 200	Jones
Amphion	1 589	Liberty
Bijou	1 562	Linden
Broadway	1 969	Lyceum
Bushwick	2 228	Lyric
Casino	1 503	Majestic
Columbia	1 673	Myrtle
Comedy	1 123	New Montauk
Crescent	1 610	Novelty
DeKalb	2 232	Olympic
Empire	1 740	Orpheum
Fifth Avenue	1 063	Oxford
Folly	1 840	Payton's
Fulton	1 492	Prospect Hall
Gayety	1 455	Royal
Gotham	958	Shubert's
Grand Opera-House	1 515	Star
Greenpoint	1 776	

ensions of Some Theaters and Opera-Houses **

the dimensions, in feet, of some of the earlier theaters in this country

cation	Auditorium			Proscenium-opening		Stage		
	Width	Depth	Height	Width	Height	Width	Depth*	Height‡
tersburg	58	76	58	56	....	75	84	....
	51	78	47	41	....	92	76	....
	71	95	64	49	....	86	78	....
s	74	73	83	52	....	66	74	....
	47	56	57	37	....	80	69	....
Paris	66	76	66	43	....	78	82	....
London	51	66	....	32	....	86	55	....
don	56	64	....	32	....	48	80	....
	....	71	58	46	....	87	68	....
usic, New	62	87	....	48	....	83	71	....
ic, Phila-	66	78	74	48	....	90	72	....
	60	65	....	30	....	62	38	....
	68	61	....	31	....	68	46	....
ra-House,	....	....	....	54	50	100	73	88
Chicago	....	....	....	....	....	110	70	95
rk	69	66	....	34	34	67	30	65
New York	70¾	79	47⅔†	35	34	40	65½	....
rk	56⅙	52	....	27	....	71½	28⅓	....
ew York	....	....	....	....	....	80	35	....
York	74½	74½	....	39	39	77¾	43⅓	73½
re Palace,	74½	74½	....	34	34‖	70	40	70
rk	67¼	67	....	32	30	67¼	30⅔	....
use, Cin-	67	69	....	36	34	67	41	....
oston¶	79½	85½	70†	40	34	68	45½	....
	77	80⅔	....	....	....	60	42	70

ter-Dimensions.†† "The utmost distance from the front of ear ought not to exceed 75 ft, or the limit the voice is capable lateral direction."

n the curtain-line, the San Carlo Theater in Naples is 73 ft; the

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ain or back line of proscenium opening.
n stage to center of ceiling.
on" or rigging-loft.
in 1893.
ed to 40 by 40 ft.
is theater is in the shape of a horseshoe.
ith table of Seating Capacities of Churches, Theaters, etc., page 1574.
lating to recently constructed buildings of these types.
nning and *Construction of American Theaters*, by Wm. H. Birkmire.

theater at Bologna, 74 ft. Of the London theaters, the Adelphi is 74 ft, Cov Garden 80 ft, the Gaiety 53 ft 6 in, Lancaster 58 ft 4 in, Marylebone 74 ft the Globe 47 ft 6 in."

The width of the ideal theater, between inside walls, should be from 7 75 ft, and "the ceiling should be from 55 to 65, or even 70 ft above the sta level."

"The depth of the parquet-floor at the orchestra-rail is governed by t stage-level, and is generally from 3 ft 6 in to 4 ft 3 in below the stage. A dep of 3 ft 9 in is a good height, as it fixes the eye of the spectator 5 in above t stage-level."

"The height of the stage, that is, from the floor to the bottom of the 'gridir or rigging-loft, should be 2 or 3 ft over twice the height of proscenium-open in order that the fire-curtain may be raised the full height of the open There should be a height of 7 ft above the gridiron to enable the fly-men to adj their ropes with facility.

Proportioning Gutters and Conductors to the Roof-Surface. The si of gutters and down-spouts and their distance apart for roofs of mill-build with a ¼ pitch and of different spans are shown in the following table:*

One-half roof-span, in feet	10	20	30	40	50	60	70
Size of gutter, in inches	5	5	6	6	7	7	8
Size of down-spouts, in inches	3	3	4	4	5	5	6
Spacing of down-spouts, in feet	50	50	50	50	40	40	[illegible]

The specifications of the American Bridge Company provide as follo the size of gutters and conductors: †

Span of roof	Gutters	Conductors
Up to 50 ft	6 in	4 in every 40 ft
From 50 to 70 ft	7 in	5 in every 40 ft
From 70 to 100 ft	8 in	5 in every 40 ft

Hanging gutters should have a slope of about 1 in in every 16 ft.

"The Produce Exchange Building in New York City, with a roof-are three-fourths of an acre, roughly speaking, has twelve leaders, each about in diameter. The roof, which is paved with fire-brick, is graded with slope perhaps one in fifty toward the points at which the leader-openings are plac most of these draining-surfaces being about 40 by 70 ft each. The p here made is equivalent to about 1 sq in of leader-opening to 140 sq ft of surface. On the Sloane Building, at 19th Street and Broadway, New Yor City, with a roof-area of 18 000 or 20 000 sq ft, sloping one in twenty-five, the are two leaders, each about 6 in in diameter, and a third rectangular lea 4 by 6 in in cross-section. This gives an allowance of 240 sq ft of surface to the square inch of leader-opening, while on the Massachusetts Hospital Life Ins ance Company's Building and the Hemenway Building, in Boston, the propo is only from 60 to 70 sq ft to the square inch of opening." ‡

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* H. G. Tyrrell.

† M. S. Ketchum.

‡ Dwight Potter in The Technology Quarterly.

ELEVATOR-SERVICE IN BUILDINGS*

eral Considerations. An efficient elevator-service may be obtained by
es of any one of several types, the choice of the one decided upon for any
g depending upon varying conditions. The following is a general classi-
of elevators (see, also, page 1588):

ydraulic elevators:
(1) Vertical, geared hydraulic type.
(2) Horizontal, geared hydraulic type.
(3) Direct-lift plunger-type.
(4) Inverted (high pressure) plunger-type.
lectric elevators:
(1) Drum-type.
(2) Worm-gear traction-type.
(3) Helical-gear type.
(4) Gearless, traction-type.
    (*a*) Direct-drive (one-to-one) type.
    (*b*) Two-to-one type.

ldition to these, there are also the BELT-DRIVEN type of elevators, and the
OWER elevators. The belt-driven type may be either SINGLE-BELT or
-BELT driven, the former being used with a reversible motor and the
vhere driving-power is taken from a line-shaft. In view of varying and
nes conflicting claims of competing manufacturers, the architect's de-
must be governed by impartial engineering judgment rendered after a
study of the problem in each case.

red Versus Gearless Types of Elevators. (See, also, page 1589.)
has been much discussion regarding the merits and demerits of geared
arless machines for elevators and the efficiency and future of each.
acturers of gearless traction-machines have claimed that the use of the
gear, for example, for elevator-machines, being a relatively recent devel-
t, has not extended over a sufficient length of time to permit of extensive
nite data; that they are used for different and less severe service than
r which the gearless traction-machines are employed; and that they can-
al the gearless traction-machines from the standpoint of efficiency. On
er hand, the manufacturers of helical-gear elevator-machines claim that
ave been in successful use for many years, the substitution of helical
or worm-gears being the only difference made in the application to their
elevators; that the helical-gear elevators installed in some of the highest
uildings are doing as much work as any gearless traction-machines; and
e mechanical efficiency of the helical-gear machines is only a little below
the gearless traction the electrical efficiency under local or ordinary
conditions, greater, and the car-mile consumption in kilowatt-hours, less.

stions of Cost and Efficiency of Elevators. The principal demerit
tor-machines of the gearless type is their relatively high first cost, al-
even that is much lower than the initial cost of elevators of the plunger-
The use of any gear, whether of the helical, worm or spur-type, is, in the
of many engineers, to be recommended only for the purpose of obtaining

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matter relating to elevators and elevator-service is condensed and adapted by
on, from data furnished by various engineers and manufacturers, and papers
tis Elevator Company, New York City; The H. J. Reedy Company, Cincinnati,
. P. Bolton of The R. P. Bolton Company, Consulting Engineers, New York,
hor of "Elevator Service"; C. E. Knox, Consulting Engineer, New York; M. W.
Consulting Engineer, New York; and others.

a higher-speed motor, because a higher-speed motor costs less than a low motor. The helical gear is generally considered a more efficient type o than the worm-gear and has a deserved place in the development of the tor-industry. The helical-geared traction-elevators will undoubtedly tensively used, for the reason that, even if they are not considered by engineers to be as good in some details as machines of the gearless, t type, they are less expensive. It is undoubtedly true, however, that the duction of any gear means some loss in power, and it is claimed that tes that low-speed motors can be designed which are in every way as effic high-speed motors. The data and statements in the following paragraph ing to elevator-service in buildings are presented as useful aids to archite include some opinions and conclusions which are to be accepted or mod the light of constant additions to engineering knowledge.

A. General Conditions Affecting the Requirements of Specifica for Elevator-Service *

Electric Versus Hydraulic Elevators. The question of the type o tors, whether electric or hydraulic, is best determined by the local condi the special conditions which exist in every plant. The relation of the e equipment to the entire mechanical equipment should be carefully co and should be decided only after mature deliberation and consultati unprejudiced engineers and elevator-manufacturers. At the present tim about 90% of the elevators being installed are electric, and this inc types of buildings from the small one with but one elevator to the scrapers of the big cities. The electric equipment recommends itself, it has all of the safety-features of the hydraulic equipment, it is a more system, is more adaptable to all kinds of conditions, and requires much le The question of space is a particularly important consideration in office-b Furthermore, the control-system is more automatic, the acceleration and tion of the car can be made more rapid, and the stops more accurate; ciency, also, is higher and in most cases the cost of operation lower. (paragraph on Comparison of Merits of Electric, Traction and Hydraulic, Elevators, page 1590.)

Location of Hoistways and Machinery-Room. The location of t ways is rather a matter for the good judgment of the architect. In t equipment all elevators serving one portion of the building and for t character of service, should be placed in one BANK and not distributed rated. Thus, all express-elevators should be together and in one bank, a also, the locals. The hoistways should be so placed that the entranc stories, are on the same side of the car. In some of the larger cities, trances for a passenger-car are not permitted, unless the doors can be c by the attendant without leaving the operating-device. The machin should be well ventilated, light and clean as possible, in order that the m may be given proper attention. This room should also be large enough the machines readily accessible for repairs and inspection. Where the have heavy parts, which it may be necessary to remove from time to repairs, it is advisable to locate a trolley-beam with hand-hoist above facilitate the handling of these parts.

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* The Otis Elevator Company, New York, has been of assistance in furnis of the engineering data of Section A, of this article on elevators. Among ot it considers especially those conditions which should be considered and ma by the architect preliminary to the preparation of the elevator-specificati paragraphs of Section A should be read in connection with those of Section B, and the data compared.

ınd Sizes of Elevators. (See, also, pages 1593 and 1595.) The
izes of elevators are governed by the following considerations: (1)
of the building, (2) the height of the building, (3) the rentable
time-intervals between the departures of cars, (5) the number of
erved, (6) the average number and length of stops per trip, (7)
the elevators and (8) the type of elevators used. No iron-clad
given for all types of buildings, but the larger office-buildings,
or light-manufacturing buildings have been sufficiently regular
varrant some general rules, based upon experience; even in these
r, the governing conditions vary with the size of the building.
ıost essential requirements for a satisfactory plant is QUICK SER-
first-class office-buildings the intervals between cars should not
onds. The number of stories to be served by a car should be a
For example, in a fifteen-story building, assuming that stops are
of the stories for one passenger each, and allowing 2 sq ft for each
d 4 sq ft for the operator, the car should have an inside area of
In order to facilitate unloading and thus increase the efficiency
, it is desirable to have the width of the car greater than the depth.
case, a car with outside dimensions of about 6 ft wide by 5 ft deep
e best results, showing a difference of from 15 to 20% between the
dth. In specifying the equipment, it is better to call for several
d cars and a high speed, than for a few large cars of slower speed
acities. Thus, three cars, each carrying one-third of all the passen-
r than two cars, each carrying one-half, as the service is increased
e period between cars less. As the elevator-service largely deter-
cess of a building, it is of vital importance that a sufficient number
e installed to handle the regular traffic, as well as emergency-con-
e of a shut-down. To illustrate what is considered the proper
passenger-elevators for buildings of various heights, the following
, based upon a rentable area of 8 000 sq ft per story and 125 sq ft
This table shows the various combinations for elevators with a
400 to 500 ft and of 600 ft per min for buildings of from 10 to
ve the ground.

Table Showing Number of Elevators Required

Express oo ft per min	Local 600 ft per min	Express 600 ft per min	Local 500 ft per min	Express 500 ft per min	Local 400 ft per min
....	4	....	5	....	5
....	5	....	5	....	6
....	6	....	7	....	7
....	7	....	8	....	8
press to 11 5	1 to 11 5	Express to 10 5	1 to 10 5	Express to 11 5	500 ft per min 1 to 11 5
[illegible]	All locals 8	....	....	....	....
press to 12 5	1 to 12 5	Express to 11 5	1 to 11 5	Express to 12 6	1 to 12 6
press to 15 6	1 to 15 6	Express to 14 6	1 to 14 6	Express to 15 7	1 to 15 7
press to 18 7	1 to 18 7	Express to 17 7	1 to 17 7	Express to 18 8	1 to 18 8

Buildings equipped with both local and express-service should
number of elevators for each class of service. In the case of t
story building for 600 ft-per-min speed, it is to be noted that the
are shown serving from the first to fifteenth story, whereas the e
serve from the fifteenth to the twenty-fifth story. The expres
not serve as many stories as the locals on account of the extra ti
the run to the first express-landing. With the distribution as sh
for all stories is about equal, and both express-elevators and
operate on about the same schedule. In the fourth and fif
shown what is considered the best arrangement with the e
operating at a 600 and the locals at a 500 ft-per-min speed. U
with the second and third columns, it will be noted that the e
are to serve one additional story. This is due to the difference i
the express-elevators and local elevators and is done so that th
still remain the same for both. (See, also, paragraph on the Lo
Round-Trip Time, page 1595.)

Loads and Speeds. The sizes of the machines or hoistin
determined by the loads and speeds. The loads for passenge
figured on a basis of 75 lb per sq ft of inside area of platform.
very important factor, as the foregoing indicates. This is us
local ordinances, and in New York City, cars stopping at all sto
mitted to exceed a speed of 500 ft per min. For express-service
of the shaft where no stop is made, a speed of 700 ft per min i
NO-STOP DISTANCE must be at least 80 ft or more. The best c
vator-insurance will not permit electric-drum elevators for a
400 ft per min, whereas the gearless, traction-drive type and the
are approved up to the limits, as noted above. In hydraulic pla
to specify the number of round trips per hour for the entire ele
This is required in order to determine an adequate pumping-pl

Hoistways. The hoistways should be finished to plumb-lin
that the car running on guide-rails set to plumb-line will at all
same clearance. Supports should be provided adjacent to the
overhead beams at a distance, if possible, of at least 4 ft from
frame when the platform is flush with the top landing. This
be increased where possible so that the car will have ample
preventing accidents due to striking the overhead work, in ca
past the top landing. The minimum clearances between the
frame and the overhead apparatus are usually limited by the loc
lations, and these should be consulted. In the case of the ele
at a speed greater than 350 ft per min, the distance given above
have to be increased in order to comply with these regulations.
provided at the bottom of the shaft. This should be at least
is the case with the overhead clearances, the depth is usually
building regulations, in accordance with the speed of the elev
possible, the hoistways should be so planned that the main gu
placed at the sides of the car. Supports should be provided at
these rails, and where the distance between floors is greater t
mediate supports should be provided. The distance from the
overhead beams, to the penthouse or skylight-roof, varies with
stallation, but can be accurately obtained from the elevator-

Protection of Counterweights. In New York City the Bu
requires that where the counterweights run in the same sha
must be protected with a substantial screen of iron from th

w, except where the plunger-type or traction-type elevator is

vs Governing Elevator-Installation. The Bureau of Build-
f Manhattan, New York City, issued regulations * governing
, inspection and operation of all types of elevators, and the
of all architects is called to them, as they are not only obligatory
cellent guides to practice at all times. The foregoing paragraphs
give an idea of what the architect must consider and provide
r the reception of the elevator-apparatus, and what he must
der to enable the manufacturer to intelligently design and lay
ry.

esigns and Special Apparatus. The specifying of apparatus
uction is, as a rule, not to be recommended. Standard designs
s much as possible, as (1) they are more apt to be well designed,
, (2) they are undoubtedly less expensive, both in initial cost
e and (3) repair-parts may be more easily and quickly obtained

s for Elevator-Installation. The specifications should include
the following classification.

service and number of elevators of each service.
m load wanted.
m speed.
h maximum speed.
m number of round trips per hour for each elevator.
of control. For electric elevators, car-switch control should
d for passenger-service and for all elevators for a speed over
per min.
oistways and area of car-platforms.
f car-platform in feet, number of car-landings, and number of
gs at each landing.
sed. If electric, direct or alternating current, the voltage and,
he phase of cycles for alternating current should be given. If
lic, the steam-pressure or electric current characteristics for
mp-motors or the water-pressure, if the purchaser provides the
, tanks or other source of water-pressure supply.
-elevation showing landings, supports for overhead beams, space
overhead sheaves, and runbys at top and bottom; a sketch-plan
g size and shape of hoistways, entrances, position of car and
rweight, guide-rails, and location of space available for machines,
, tanks, etc., with reference to hoistways.
counterweight guide-rails, whether of wood or steel.
for fastening the rails, character of these supports, and where
w located.
finished car or cage, that is, the specified amount to be allowed
h, the design being subject to the approval of the architect.
and size of ropes, if not left to the judgment of the elevator-
ctor. The largest sheaves possible should always be required,
factor determines largely the life of the ropes.
f signals, that is, (*a*) annunciators in the cars with push-buttons
landings, (*b*) UP and DOWN signals in the cars, with UP and DOWN
s at the landings, so arranged that a car going up receives only

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show the first available car in either direction.

(17) Source of power. It should be specified whether
brought to the elevator-apparatus by the pu
vator-contractor. If by the latter, a sketch s
the distance, and for the electric system the sp
whether the wiring is to be open, that is, on c
conduits; the sizes of wire, and the switches,
hydraulic system, the size of pipe for steam-s
The sizes of water-piping should be left to the
he should be held responsible for them. Als
draulic system operating from street-mains, t
state by whom the piping is to be done and wh
meter.

(18) Pumps and tanks in hydraulic plants. These sh
contractor. The specifications should state
to be just ample to do the work, or whether
capacity, with reserve-units, to provide again
service in case of accident to a pump or tank,
but the sizes and design should be left to the ju
elevator-contractor.

(19) Foundations for the machine, whether they are
purchaser or by the contractor.

(20) Miscellaneous. Gratings underneath the overhe
ing in addition to the standard factory-finish
tioned above are generally furnished by the pu
contracts, but this should be clearly set forth
cations.

Safety-Devices for Elevators. (See, also, page 1
safety-devices cannot be too carefully considered for
passenger-elevators in particular only the best and
apparatus should be installed. Each car should be
chanical device designed to grip the rails and stop the

ly is interrupted. The brake should be released electrically and ns of spring-pressure. Automatic limits should be placed at tom of the hoistway, to automatically slow down and stop the of travel, independently of the operator.

action-Elevators.* Among the more recent developments of ustry is the electric, gearless, traction-elevator (Figs. 1 and 2). 5.) The designing of an efficient slow- de it practical to build a traction-machine -sheave mounted directly upon the armature-shaft, thus eliminating the use of gears to reduce to the desired car-speed. This gearless machine is used for speeds from 250 ft per min and above. The manufacturers of this type of machine claim that it is the outcome of a general tendency toward simplicity in design with efficiency in operation. The machines are generally located over the hatchway. The car is supported by cables which lead from the car directly over the driving-sheave, with overhead installation, then partially around the auxiliary idler or leading-sheave and again over the driving-sheave to the counterweight. With this arrangement a complete turn around the driving-sheave and the idler-sheave is obtained, giving sufficient tractive effort to drive the car. The machine being placed overhead, the cables can lead directly to the car and counterweights; and as this allows the cables same direction, it is claimed by the manufacturers that ge and that the life of the cables is appreciably lengthened. are used for connections to the car and counterweight to twisting effort due to the reaving of the cables. As soon or counterweight is obstructed, the tension in the cables is onsequently the tractive effort reduced. This arrangement, rings either the car or counterweight to rest and prevents limits of travel, and into the overhead beams, should either the buffers at the bottom of the shaft. Underneath both car ht are placed oil-buffers designed to bring the car or counter- the limits of travel, from full speed. At the top and bottom the car is stopped automatically by a series of electric switches. these switches is so timed that the car is brought to a smooth . The slow-speed shunt-motor, with its control, makes a flex- ne acceleration and retardation may be arranged to suit the -requirements. For speeds below 450 ft per min, it is the prac- e slow speed by passing the cables around sheaves mounted in

for
on-

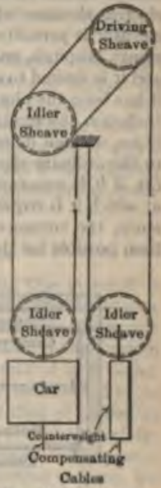

Fig. 2. Roping for 2 to 1 Traction-machine

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valuable data relating to the relative advantages of the helical-gear ired with those of the traction-type, see papers published by the H. J. Cincinnati, Ohio, and others advocating the geared machines. Editor.

the cross-head of the car and of the counterweight, and anchoring t the cables at the top of the hoistway. These sheaves, with their bal are specially made to withstand the heavy service to which they are In addition to the above details, elevators of this type should be pro all of the regular safety-devices used with passenger-elevators.

Electric Elevators with Push-Button Control. One of the most and serviceable developments in the elevator-industry is that of the electric elevator with push-button control. In New York City th elevator is permitted only in residences, but in other cities it is use ments, hospitals, and other places where the service is very light and in and it is desired to dispense with an attendant. In the design of thes it has been the aim to provide all safety-devices and appliances to m stallation absolutely safe, so that the elevators may be operated even alone, without danger. In each story is located a button, similar in to the ordinary signal-button, and the passenger, by pressing this, m car, if it is unoccupied or not in use, to any story. The car comes t at which it is required, and stops automatically. When it comes to story, the entrance-door to the hoistway is automatically unlocke then possible for the passenger to open the door and the car-gate, an

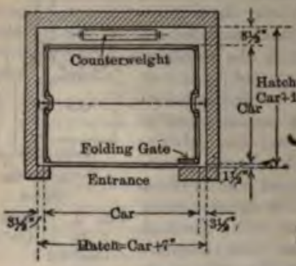

Fig. 3. Standard Hatchway and Car-platform. Side-guides

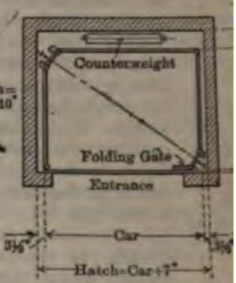

Fig. 4. Standard Hatchway platform. Corner-gu

car. The hoistway-gates and the car-gate are so arranged that t inoperative until both are tightly closed. The hoistway-doors c from a hall, only when the car is at the landing of that particular car is a bank of buttons corresponding to the various stories se a stop-button or emergency-button. After entering the car, an hatchway-door and the car-gate, the passenger can push the car corresponding to the story to which he desires to go. The ca to the designated story and stop automatically. Should the pa for any reason, to stop the car at any point of its travel, he can do the stop or emergency-button. The car is in the complete cont senger, as, after the initial operation of calling or sending it to further operation cannot be interfered with until after both the and the car-gate are opened and closed. This means that no ot call the car until after the passenger has reached the desired la car, and closed the gate and door. In some equipments for elevat the device for releasing the door-lock is prevented from operatin is in motion. This is a very desirable safety-feature, as other

y released as the car passes up or down the hoistway, and a person
can open the door during the momentary period that it is unlocked.
s the gate on the car is omitted; but this is a very dangerous prac-
ld not be permitted. Elevators of this type are designed for oper-
lirect current or alternating current, and single or multiphase
gle phase should be avoided, if possible, and before deciding upon
current, the consent of the electric power company should be ob-
acing upon their lines a motor with the heavy inrush of current
arting.

Relations of Hatchway, Platform and Car-Sizes. (See, also,
In Figs. 3 and 4 are shown some typical elevator layouts for
llations, with side and corner-posts and steel construction. (See,
The clearances shown are for elevators traveling at a speed of
n or more, and may be reduced about 1½ in for elevators of slower
of the minimum dimensions given with Figs. 3 and 4 vary slightly
ven with Fig. 7 and in Table D, page 1596, but agree in the essen-
ents.

B. Electric, Passenger-Elevator Systems *

Development. The object in view in presenting this material is
all the details of elevator-construction or the mechanical features,
the results of a study in connection with the economic division of
vators and an efficient elevator-service for the traffic of the modern
r distinct type of office-building. The requirement of such build-
y ample and adequate elevator-service, not only because the mon-
f the building may otherwise be affected, but in time of necessity,
re or other panic, the occupants must be readily brought to safety.
arly development of the sky-scraper the necessity for a proper
ice was partly overlooked, and perhaps not altogether realized, for
older buildings suffer from a lack of traveling-facilities, resulting
nience to the many occupants. The tenants of the upper stories
obliged to wait on the up trip of the elevator, and the people occu-
er portion of the building are left behind on the down trip.

nsive Use of Elevators. To fully indicate the extensive use to
vator has been adopted for passenger traffic in large cities, the in-
Borough of Manhattan of New York City is given. There were
t 10 000 machines in service, twice the number that were in oper-
, and these were divided among the different classes of buildings
y as follows:

levators in office-buildings over 10 stories high.
levators in office-buildings under 10 stories high.
levators in loft-buildings.
levators in residences.
levators in apartment-houses.
levators in department and other stores.
levators in hotels, clubs, institutions, etc.

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r in Section B of this article on Elevators is, by permission, condensed
om data contained in papers by M. W. Ehrlich, consulting engineer. The
eared in the April, May and June, 1914, issues of Electrical Engineering,
were published in condensed form in Lefax, by the Standard Corpora-
lphia. Section B includes a brief outline of elevator-development, some
derations and some installation-data, and the paragraphs of this Section
in connection with those of Section A, page 1580, and the data compared.

Besides these passenger-cars, the buildings requiring freight-service inv an additional 10 000 machines.

Two Common Types of Elevators. In modern elevator-practic are but two common types of successful machines in use, the hydraulic a tric elevators. These may both be subdivided in the classification acc

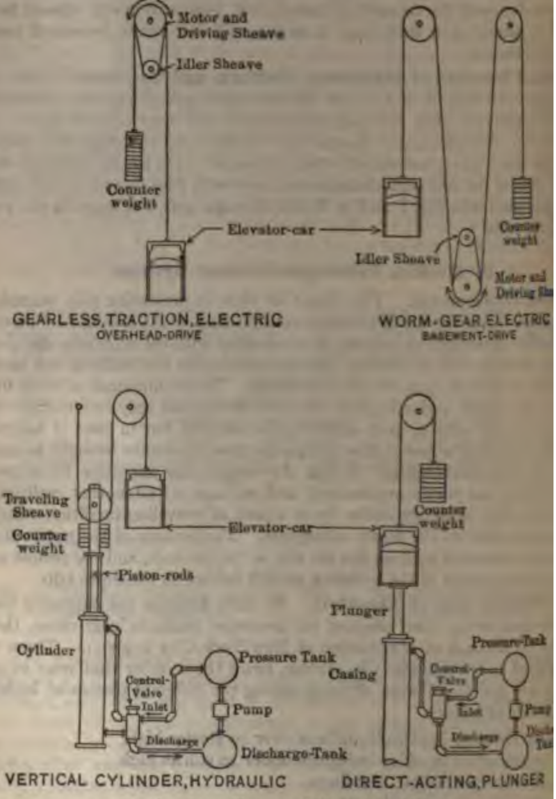


Fig. 5. Some Types and Varieties of Elevators

to the mode of drive or operation and the transmission of power, thereby sh an apparent variety of elevators. The machine of the hydraulic type m of the vertical-cylinder pattern or of the plunger-type, while the electrical ratus may be of the drum, worm-gear or gearless traction-type. Some types and varieties are illustrated in Fig. 5. (See, also, Figs. 1 and 2, pag and general classification on page 1579.)

A Short Historical Account of the Development of the Comm Passenger-Elevator brings one back a little more than half a century

f the first STEAM-ELEVATOR. This form of drive was soon replaced
-BALANCE type of hydraulic elevator, which, even though a faster
ed to be, in operation, quite dangerous. For a number of years
yed the distinction of being the only high-speed apparatus until
f the VERTICAL-CYLINDER HYDRAULIC ELEVATOR, about twenty
Running-speeds as high as 400 ft per min were readily attained,
nt of the ease in handling and the safety in operation, these ele-
ined favor and were the only types of machines installed in the
lings. The electric DRUM-MACHINE made its first appearance in
ring the year 1889, and owing to the merits of this new system,
achine soon established itself as a successful competitor with the
e. The only obstacle remaining was to overcome the slower
his brought out the Sprague LONG-SCREW, ELECTRIC ELEVATOR.
this type proved quite costly to maintain and operate, but on
eir possibilities of speed and high rise, were installed in several
s. These different types of elevators helped considerably in the
of the sky-scraper buildings, and as further building projects
a extension in height, a hitherto unknown condition of passenger-
ice had to be met. About the year 1900 the DIRECT-ACTING
RAULIC ELEVATOR was introduced to fulfil this increasing demand
high rise with high speed. The inherent safety in operation and
high economy allowed for no doubt as to the possibilities of the
after several years, experience pointed out that the advantages
lic plunger-elevator were somewhat limited in certain directions,
er conditions of a rise not exceeding 150 ft could the character-
afe and economical plunger-elevator be maintained.

nd Geared Elevators. (See, also, page 1579.) Recent experi-
ted to perfect an electric elevator that would meet the growing
of heavy passenger traffic in the newest form of tall office-build-
ulted in the production of what is now commercially known as the
r GEARLESS TRACTION-ELEVATOR. Among the earliest New York
f this type of electric elevator may be named those in the Singer
Tower, and later those in the Metropolitan Building and Tower,
st developments include the Woolworth and the Equitable Build-
paratus used in the Municipal Building is one in which the
an adaptation of the usual double-worm-and-gear drive between
igh-speed motor and a cable-drum, a double set of intermeshing
ing employed between the two gear-shafts. In summarizing, it
to mention that the commercial or useful life of an elevator and
mechanisms seldom exceeds fifteen years, and that where remod-
resorted to, the ELECTRIC DRUM and WORM-GEAR TRACTION have
substituted for the HYDRAULIC TYPE in buildings not exceeding
to sixteen stories in height; and that in higher structures the
ion-elevator or its modification in the form of an electric TWO-
ION-ELEVATOR has been resorted to.

Electric and Hydraulic Elevators. (See page 1584.) It is
, that both the electric and hydraulic types of elevators have
d to a state of high efficiency, and they may, therefore, be used
fety. Of the hydraulic types it may be said that the plunger-
inherently safer than those which are suspended, or than even the
electric traction-elevators; but it cannot be denied that the many
nd improved appliances attached to elevators of the various electric
de the latter as reliable as hydraulic machines designed according

to best practice. It is claimed that the electric traction-elevator is relati free from the element of danger because of the improved methods of pow transmission and the peculiar form of windings used for the drive.

Comparison of Merits of Electric, Traction, and Hydraulic Plunge Elevators. In narrowing down the question as to the merits of the TRACTION-ELEVATOR and of the HYDRAULIC PLUNGER-ELEVATOR for pass service in tall office-buildings of to-day, it might be well to note that the elevator-installations, almost without exception, have favored the ele Not only is the cost of installing the traction-machine from 25 to 15% less than that of the plunger-type, but the room occupied by the driving-machinery reduced to a minimum, and, as a matter of fact, may be placed at the head directly over the elevator-shaft. If no local supply of electricity is availab the premises, the public source may be resorted to. The difficulty with plunger-elevator for high-rise, high-speed work lies in the requirement for ing the mass of water and the massive plunger proper, and as this weight cannot be readily and smoothly stopped, the result is a sluggish starting and stopping. At any rate, it remains an open question as to wh the economic values attached to modern buildings would favor the install of the plunger-elevator, with its accompanying pumping-plant, which nec sarily occupies considerable floor-space. The choice, therefore, would tend favor the HIGH-RISE, HIGH-SPEED ELECTRIC TRACTION-ELEVATOR for pass service. (See, also, paragraph on Electric Versus Hydraulic Elevators, pa 1580.)

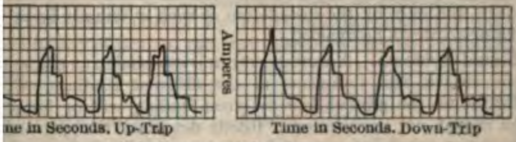
Table A. Relative Operating-Costs of Elevators

Costs	Office-building				Loft-building				Apartment-		
	Traction electric	Worm-gear electric	Vertical-cylinder hydraulic	Direct plunger	Traction	Worm-gear	Hydraulic	Plunger	Traction	Worm-gear	Hydraulic
Per cent of rentals	8.5	7.2	6.8	6.5	8.0	6.8	6.5	6.2	6.8	6.0	[illegible]
Cents per car-mile.	25	22	20	19	23.8	20	19	18	20	18	17
Dollars per car per annum..........	2 100	1 850	1 680	1 600	1075	900	860	810	560	510	[illegible]
Per cent of all operating-costs......	14.1	12.0	11.3	11.0	18.0	15.4	14.8	14.0	13.6	12.0	[illegible]

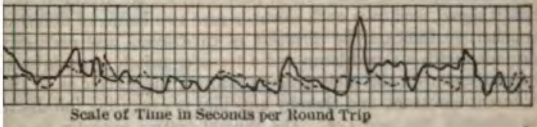
Relative Operating-Costs of Elevators. The figures given in Table may prove of interest in pointing out the relatively higher operating-costs of t different ELECTRIC types over the VERTICAL-CYLINDER HYDRAULIC and PLUNG ELEVATORS. The values given represent only the cost of labor, power, rep and supplies. By a close perusal of the amounts listed, it will be confir that the economies of the plunger cannot be utilized beneficially in tall o buildings, on account of the mechanical difficulties, and in other types of sm buildings, allowing for a low rise, the installation cost becomes exorbitant. the relatively high first cost of this type of machine were taken into consi tion, with an addition for the extra cost in building-construction necessary the space occupied by the pump and tank-equipment, the total expenditu the whole would show no great favor either way. In explaining the v given in Table A, it should be understood that the figures are computed basis of actual records of several buildings that have been brought to the neral method of comparing records in business buildings is to sts with the total annual income or rental. The total operade the expense in the mechanical, electrical and building vering all costs of labor and material for the maintenance of the ns of service. Therefore the annual cost of operating an ele- given as a percentage of the gross rentals received, and is s a percentage of the total operating-expenditure of the buildderation. The average cost in cents per car-mile traversed is ther with the average annual cost in dollars to pay for the ng and repairing, the necessary power, and the material and l per single elevator.

onsiderations. The efficient operation of an elevator-system ogether on the economic division and disposition of the cars, as ent becomes one of the main factors. It is self-evident, therervice of an elevator is limited not only by the different classes of ing, riding and leaving the conveyance, but by the experience or STARTER and his ability to understand the demands of the ersonal peculiarities of the elevator-operators.

ules. It is now common practice to dispatch the various maevator-system on a predetermined time-schedule, thus avoiding nt any confusion or overcrowding that would otherwise arise. l established that the travel of elevators under consecutive-trip on allows for a highly efficient service, not only in the handling t in the demand for power, which is thereby reduced to a min-



(*a*) Operation of one car



(*b*) Operation of a bank of elevators

Recorded Current-consumption of Gearless Traction-elevators

rams. The POWER-DIAGRAMS (Fig. 6) point to the effect of a er service under different conditions. The upper curve (a) was t-conditions and represents the operation of one elevator. The le car is approximately equal to the designed machine-balance, and down trips, and the number of stops corresponds to the per mile under actual service in office-buildings. This diagram to allow for a proper understanding of the combined curve (b), *al round-trip operation of a bank of elevators in one of the New buildings at an early-morning hour. The full or solid-line cessive power-demand due to an inconsistent* SCHEDULE, the

cars having been dispatched by a starter who may be identified as
dotted or broken-line curve shows the more expert handling under the
trips by starter *F*, the same operators running the cars in each case

Safety-Appliances. (See, also, page 1584.) To minimize the
dents in elevator-practice, a SAFETY-LOCK is recommended, so atta
will not permit the elevator to leave a landing until the gate has
Accidents are seldom, if ever, due to the faulty behavior of the ele
but sometimes the breaking of suspension-ropes, as recorded by a n
cases, will result in a serious accident. The most frequent cause
connected with the operation of elevators, is that due directly
gence of the operator in handling the doors or elevator-gates, and
avoided by the installation of the safety-locks above recommen
as has been practically demonstrated, many of the safety-applia
older installations designed to stop a falling elevator have usually
but the improved wedge-type of JAW-SAFETY, actuated by a SPE
and attached to the more recent installations, usually acts when
exceeds its normal running-speed. This generally occurs when the
safe-distance limit has been passed, and the jar occasioned by the
of the car is not altogether a pleasant experience. The serious
fatalities due to the falling of an elevator are proportionately sma
pared with the entire list, and amount to about 20% of the total,
loss of life caused by open and unlocked gates in elevator-p
accounts for the remaining 80%. The only safety-device, therefo
be called useful, as it eliminates the personal element, is a SAFE
the several automatic devices now available for this provision of
serve merit; and while some are purely mechanical, others are
trically, and only by the installation of such automatic locks wi
elevator-accidents be avoided.

Signal-Systems. A SIGNAL-SYSTEM is essential to an efficient s
matic electric-light indicators at the different landings, with
indicator on the ground-floor or street-landing, will be found
even though not the simplest. Briefly described, the system is
dynamotor supplying current for the magnets, push-buttons a
each landing one or more sets of push-buttons are arranged for b
DOWN signal, and over each elevator-gate two lamps of different
another, to indicate the direction of car-travel; and each eleva
provided with a signal-lamp and a transfer-switch or push-butto
ical indicator on the main landing informs the starter as to the
different elevators, and thereby aids him in exercising full discret
to dispatch the next car. The general system operates in a m
mately as follows: When a push-button is pressed for either d
story, it actuates a magnet corresponding to that story, which
to the operator in any approaching car, thereby indicating a wai
and, according to the movement of the elevator, further contac
the outside signal-lamps at that story showing to the waiting
approaching that floor. In a properly proportioned elevator-sy
fer-switch is seldom used, but in buildings in which the travel bec
during the rush-hours, and when an approaching car is filled
the operator may press the transfer-button and thereby signal th

Traffic-Capacity of Elevators. The TRAFFIC-CAPACITY of
its passenger accommodations must necessarily be of such pro
handle the travel of the tenants of the building and also of their
sure a proper working schedule. From a study of existing syst

e is considered adequate, it is found that the questions of BUILD-
r as related to BUILDING-AREA and elevator TRAFFIC-CAPACITY
ned into a consideration of a proper UNIT AREA for the elevator.
e determination of the MAXIMUM TRAFFIC-CAPACITY of a passenger-
ience shows that an average weight of 140 lb may be allowed for
, and as each size of car has its corresponding load at the rated
l load divided by 140 gives the maximum number of passengers an
commodate at its designed speed. In another simple computa-
ult, an allowance of 2 sq ft of car is made for each passenger. The
city of an elevator may be of interest in computing the time re-
y a building in case of emergency; but when a car of proper unit
d, this condition is taken care of. Tests have shown that the
NGER TRAFFIC of an elevator-system bears a definite relation to
f the building, and to the MAXIMUM TRAVEL, the result being that
ormula (6).

Elevators. (See, also, page 1581.) Modern practice tends to
NUMBER OF ELEVATORS required for any office-building is really
e physical aspects and conditions of that building. Wherever it
ble to use a car of large area, the number required will certainly
f that necessary when large cars are used. It is not advisable,
se any conclusions on the number of cars to adequately satisfy
tion, unless the UNIT AREA OF THE CAR is considered.

Express-Elevators. Another important consideration is the
mon in high-class office-buildings, namely, the proper service of
PRESS-elevators.

r Elevator-Service. The formulas given below are well sub-
l give economical service-conditions based on existing systems in
s of the United States. By these formulas the number of eleva-
he division of service, and their operation may be determined.

000 (1)
2 (2)
$+ 5/100)\,n$ and $Tl = (25/s + l/10)\,n$ (3)
Te and $Ml = 2\,n/7\,Tl$ (4)
$/100\,Te$ and $Cl = 115\,n/100\,Tl$ (5)
e and $pl = 300/Tl$ (6)

n the formulas are:

er of elevators required
e feet of gross building-area served
at which express-run terminates
number of stories served
of elevator, in feet per minute
round-trip time, in minutes
s round-trip time, in minutes
traveled per hour by local
traveled per hour by express
nt consumed per hour by local, in kilowatt-hours
nt consumed per hour by express, in kilowatt-hours
ngers carried per hour by local, one way, up or down
ngers carried per hour by express, one way, up or down

n Table B represent the AVERAGE LOAD AND SPEED-COMBINATIONS
hts of buildings, together with the usual AREA OF THE ELEVATOR-

CAR consistent with the standard sizes manufactured, and should be used as a basis for selecting the proper unit areas in connection with Formula (1). The many factors entering into the operation of an elevator would affect the current consumption to a considerable extent, as may be seen in Fig. 6, previously explained. But Formula (5) agrees with modern service under average operating conditions.

Table B. Unit Area, Load and Speed-Combinations

Number of stories	Car-area, sq ft	Load, lb	Speed, ft per min
8 to 13	25	1 700	250 to 350
14 to 22	30	2 000	350 to 600
23 to 30	40	3 000	400 to 600

Table C. Elevator-Installation Data

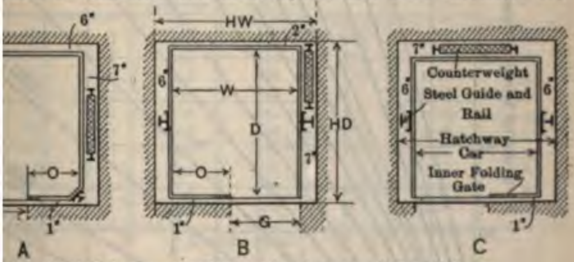
1	2	3	4	5	6	7
Building		Number of elevators required				
Number of stories	Gross area, sq ft	Total car-area, sq ft	Cars at 25 sq ft	Cars at 30 sq ft	Cars at 40 sq ft	By Formula
8	80 000	89	4	....	....	4
10	100 000	111	4	....	....	4
12	120 000	133	5	....	....	5
14	210 000	262	11	9	....	[illegible]
16	240 000	300	12	10	....	10
18	270 000	337	14	11	....	11
20	300 000	375	15	13	10	13
25	375 000	577	....	19	15	18
30	800 000	1 221	....	40	30	23

	8	9	10	11
	Round trip time in minutes			
Number of stories	*Tl* at 350 ft per min	*Tl* at 500 ft per min	*Te* at 500 ft per min	*Te* at 600 ft per min
8	1.3	....	....	....
10	1.7	....	....	....
12	2.0	....	....	....
14	2.4	2.1	....	....
16	2.7	2.4	1.6	....
18	....	2.7	1.8	....
20	....	3.0	2.0	1.8
25	....	....	2.5	2.3
30	....	....	3.0	2.7

on-Data. In order to facilitate the ready understanding of the
nulas given, Table C, embodying the computations, is presented.
headings included are numbered in respective order from 1 to 12,
explanation of the items considered will not be confusing. Under
listed the heights of buildings, with the assumed floor-areas, extend-
height of the structure, given in column 2. In column 3 are listed
quare feet of car-area now provided in many buildings of similar
nd with an adequate service. This is intended as a guide where
ations in planning the building have included a means of accom-
e standard-sized elevators most suitable for that building and where
ation has been given to the disposition of the cars. But, on the
the values listed may also be used to advantage in proportioning
of elevators required under any conditions, and where the physi-
f the building does not allow for an economic disposition of the
Any conservative unit area best suited to the conditions may then
for each car, and the number of elevators then determined. Col-
nd 6 give the numbers of cars for various standard unit areas, while
column 7 are computed by Formula (1).

al and Express Round-Trip Time for different running-speeds
olumns 8, 9, 10 and 11 of Table C, and the value for *f* as given
(2) is given in column 12. It will be noticed that in columns 8
me occupied in traversing the heights of buildings exceeding eighteen
htly more than would actually prove economical. It might be well,
point out that the speeds of local elevators for high buildings might
to advantage; but whether the service is local or express, it is not
exceed a speed-rate of 600 ft per min. In order to rectify this con-
r the speeds considered, the number of express-elevators must then
n half the total number in the system, and a subdivision of express-
er is also necessary. (See, also, Table Showing Number of Eleva-
d and notes following, page 1581.)

Hatchways and Car-Platforms. (See, also, page 1581.) The
vator-car platforms and hatchways of unit areas heretofore con-



. 7. Typical Layouts for Elevator-hatchways and Car-platforms

shown in the following diagrams (Fig. 7) illustrating three typical
dern installations with steel guide-rails. (See, also, Figs. 3 and 4.)
door-opening may be either right-hand or left-hand, as best suited
structural, or other conditions. The clear inside dimensions of the
tchway are given, and also the clearances required between this and
me of the minimum dimensions given with Fig. 7 and in Table D
from those given with Figs. 3 and 4, page 1586, but agree in the
uirements.

Table D. Sizes of Elevator-Car Platforms and Hatchways

Dimensions	Area of car-platform					
	25 sq ft		30 sq ft		[illegible]	
	ft	in	ft	in	ft	in
W = inside width of car	6	0	6	3	7	[illegible]
D = inside depth of car	4	3	4	9	5	[illegible]
O = space for operator	2	3	2	3	2	[illegible]
G = gate-opening	3	9	4	0	[illegible]	[illegible]
HW = hatch-width, car A	7	0	7	3	[illegible]	[illegible]
car B	7	4	7	7	[illegible]	[illegible]
car C	7	3	7	6	[illegible]	[illegible]
HD = hatch-depth, car A	5	1	5	7	6	[illegible]
car B	4	9	5	3	6	[illegible]
car C	5	2	5	8	6	[illegible]

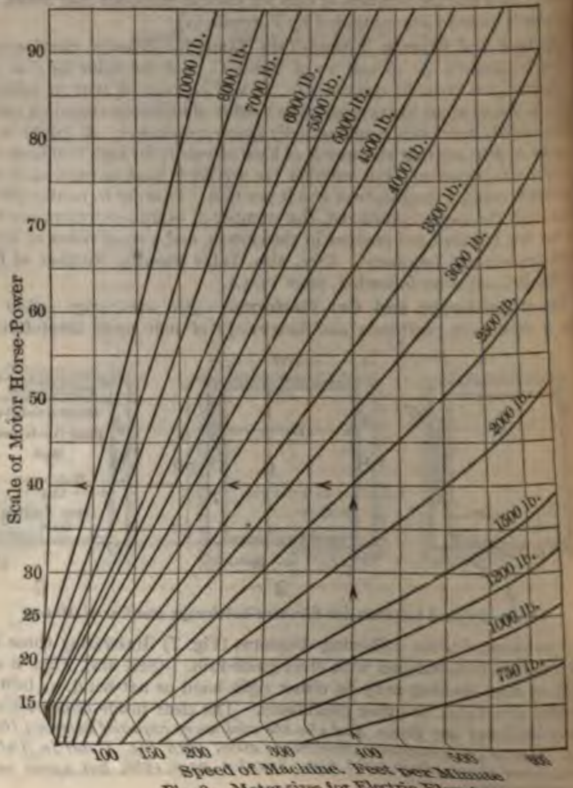


Fig. 8. Motor-sizes for Electric Elevators

otor. It is often helpful to be informed as to the SIZE OF MOTOR re- installation, and the diagram (Fig. 8) may be used for this purpose. llustration in the use of the diagram, a speed of 400 ft per min is a combined load of 2 500 lb. Following the line marked with an e speed of 400 ft, the point of intersection is then at 2 500 lb. From ow the line as indicated to the scale of motor-sizes, and the result orse-power.

Table E. Current-Consumption

Motor-size	Starting-current	Running-current
20 horse-power	102 amperes	74 amperes
40 horse-power	202 amperes	147 amperes
60 horse-power	292 amperes	213 amperes

onsumption. Table E gives the CURRENT-CONSUMPTION of motor- in elevator-practice. The figures are for direct-current motors 230 volts and are based on the results of tests.

eeders. To aid in the selection of well-proportioned ELECTRIC elevator-motors, Table F is given. The figures are for 230-volt, machines.

Vire and Conduit-Sizes for Electric Elevators, 2-Wire, 230-Volt, Direct-Current Systems

Wire		Maximum run or distance for 2% drop, ft	Conduit		
Size of each wire	Underwriters carrying capacity, amperes		Trade size for 2 wires	Inside diameter, in	Outside diameter, in
No. 3	80	154	1¼	1.38	1.66
No. 1	100	174	1½	1.61	1.90
No. 0	125	186	1½	1.61	1.90
No. 00	150	198	2	2.06	2.37
No. 000	175	212	2	2.06	2.37
No. 0000	225	226	2	2.06	2.37
No. 0000	225	226	2	2.06	2.37
00 000 c.m.*	275	248	2½	2.46	2.87
00 000 c.m.*	275	248	2½	2.46	2.87
00 000 c.m.*	325	272	3	3.06	3.50

* Circular mils.

MAIL-CHUTES

escription. This system of mailing letters by means of a specially hute connected with the receiving-box at the bottom, has come neral use in public buildings, office-buildings, apartment-houses hat the restrictions affecting the same and what is required in the *ration should be known to architects. The system is installed by under regulations of the Post-Office Department* governing its construction and location, and for this reason it is well to consult the make before permanently locating the apparatus on the plans. It may be placed any building of more than one story, used by the public, where there is a f delivery and collection-service, in the discretion of the local postmaster, subj to whose approval the contracts are made.

The Chute and Receiving-Box. The chute is required to be made with removable front and a continuous, rigid, vertical support is absolutely necess It must be of metal, its front must be of plate glass, and it must bear the insig prescribed by the department; and the whole apparatus, when erected and t Government lock put on the box, passes under the exclusive care and cont of the Post-Office Department, and the chutes become a part of the receivi boxes. These boxes may be of various patterns and highly ornamented and furnished by the makers in connection with the chutes. The work of prepar a rigid support for the chute and cutting and finishing the openings in the fl is of the utmost importance, and details showing the usual arrangements always given.

Preparatory Work. The requirements for what the manufacturers PREPARATORY WORK include a flat, vertical, continuous surface not less t

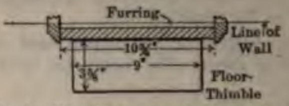


Fig. 1. Wooden Support for Mail-chute

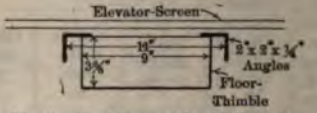


Fig. 2. Steel Support for Mail-chute

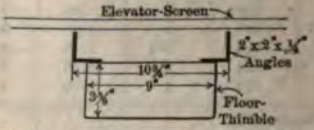


Fig. 3. Alternate Steel Support for Mail-chute

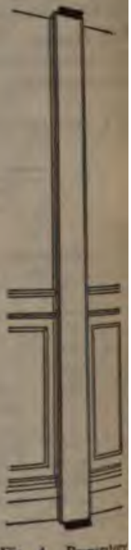


Fig. 4. Preparatory Work Complete f Mail-chute

10½ in wide, extending from the floor of the ground-story to a point 4 f above the finished floor in the top story, and an opening in each floor di in front of and centered upon this surface. These openings are neatly fini and their size and shape determined by setting in them thimbles of iron

[*] The Cutler Mail Chute Company, Rochester, N.Y.

nd delivered by the patentees, as part of their contract. In ıtions a casing of wood, suitably molded and finished to match uilding, answers every purpose. Such a casing is shown in plan, opening finished by the iron thimble. In buildings, or some- tories, where a more elaborate finish is desired, marble is sub- d, the form and construction of the casing being adapted to the course without disturbing the size and form of the front surface. ısed where the use of wood is objected to, or where it is necessary in front of an elevator-screen, or in other locations where a solid able to support the casing. Steel square-root angles, 2 by 2 by are generally used, and set as in Fig. 2, but sometimes, where fill up the space between them and the elevator-screen, they are 'ig. 3. The angles are usually bolted to the beams, and in any ıightened so that they are without twists or kinks, and the sur- ves the mail-chute plumb and flush in all stories. Fig. 4 gives f the mail-chute casings and floor-openings ready to receive the es. This work of preparing the building, except the cutting or e necessary openings in the floors, is now usually included in the act, as it has been found for many reasons undesirable to sepa- cessary openings in floors, and all patching around such open- included in the mason's or other proper specifications.

ints to be remembered are (1) that no bends or offsets can be fall being absolutely essential, and (2) that the entire apparatus to view and must be accessible, that is, it is not permitted to behind an elevator-screen or partition or through any part of ept a public corridor.

REFRIGERATORS *

uirements. The following information is given as a guide to viding for refrigerators in large residences, hotels, clubs, hospitals ıtions. Consultation with a reliable refrigerator-builder, how- lesirable before deciding upon spaces to be occupied by refriger- ng-rooms, etc., as a satisfactory refrigerator cannot be adapted portioned space. (See, also, Design of Refrigerators, under igeration, page 1611.)

efrigerators. Care should be taken to select a refrigerator in operation and easily cleansed, as modern sanitary science has ess to faulty refrigeration. Thorough insulation is an important gerator, as upon this depends economy in the use of ice and the intaining of the low temperature necessary to the proper preser- Fig. 1 shows a kitchen-refrigerator for use of families of ordi- ice-compartment is located in the middle division. The depth ore than 3 ft nor less than 2 ft, and the height may vary from The length of the front largely determines the capacity and n about 4 to 7 ft. Fig. 1 shows, also, a most satisfactory method the outside-icing feature which consists of a double outside ete, with frame and jamb. This is provided by the refrigerator- rough opening furnished by the owner in the outside wall of

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and the drawings relating to this subject were furnished the author and tt *Refrigerator Company, Buffalo, N. Y. Practical data were furnished, wick-Balke-Collender Company, New York City.* There are numer- *ms whose refrigerator-work has the highest reputation.*

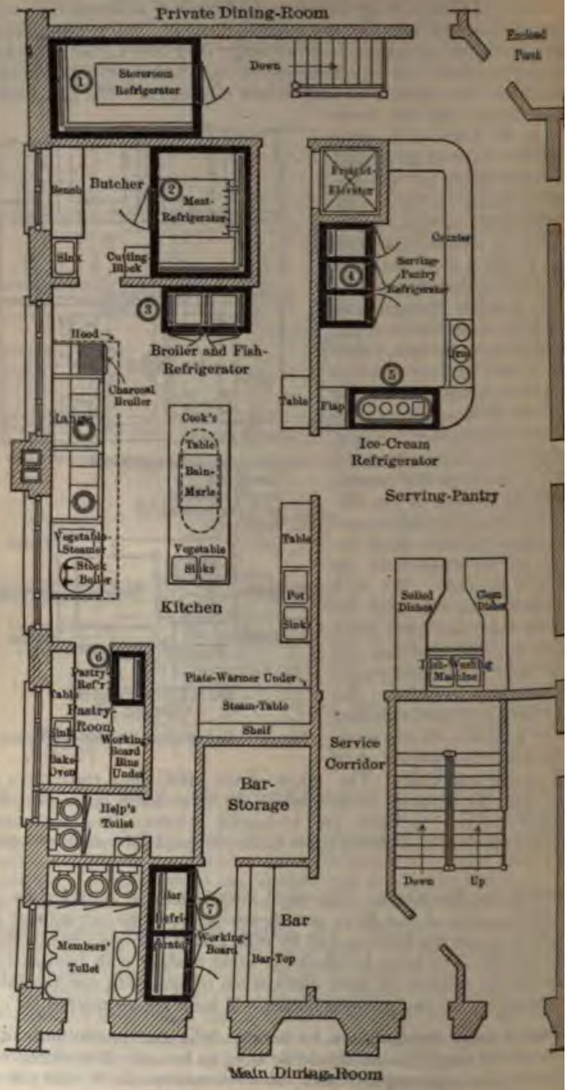

Fig. 3.* Plan of Refrigerators for Large Club-house

* The Jewett Refrigerator Company.

ugh half-height doors, hinged covers, drawers, etc., should be
n sanitary cement platform finished with cove to floor of building.
tors should not be higher than 6 ft 6 in unless provided with over-
coil-compartments, in
height should be from

(See, also, The Value
sulation, page 1610.)
n modern hotels, clubs,
tc., are insulated with
andard corkboard, the
tors being constructed
oughout, in two courses
ss, and with all joints
is applied to adjacent
uilding with Portland
thick, and this cement
in applying the inner
to the outer course in
ons and ceilings. All
flooring is asphalted
Interior finish may be
ment throughout or of
ets on walls and ceil-
rtland cement on floors.
and ceilings may be of
lain or white plate glass,
of tile, all depending
le and character of the
e equipped. The in-
aller refrigerators con-
exterior course of 7/8-in
rooved lumber, (2) two
water-proof insulating-
) a 3-in thickness of
two 1½-in courses, all
roken. To this insula-
the interior lining.

Refrigerators. Mor-
tors should be cooled
l refrigeration, the coils
longitudinally on both
nortuary-trays. Fig. 4
ortuary-refrigerator for
This may be used as a
ing mortuary-refrigera-
capacity, or the height
ced to 5 ft and the
in two instead of three
s. Mortuary-refrigerators sometimes have both fronts finished
with doors *so that bodies are accessible for identification or
om both fronts.*

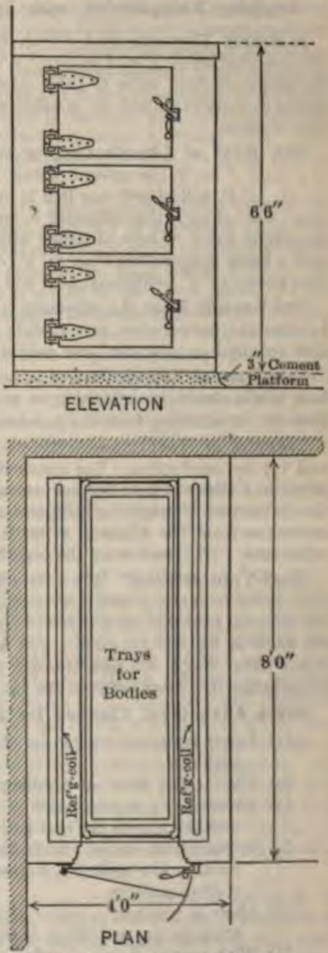

Fig. 4.* Mortuary-refrigerator

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* *The Jewett Refrigerator Company.*

MECHANICAL REFRIGERAT

A Brief Description of Methods in Common Us
Applying Refrigeration, with Special Referenc

A British Thermal Unit, (Btu), is the quantity of
the temperature of 1 lb of water 1° F. Heat used in th
the temperature of water or other substance, is said to
stance as SENSIBLE HEAT, or, in other words, heat, the p
feel, or sense.

The Heat of Liquefaction, or so-called LATENT
of a mass of ice, is the amount of heat it will absorb in
ice at 32° F. will absorb 144 Btu in melting to water
into a cake of ice is thus absorbed in melting the ice and
as LATENT HEAT, or heat absorbed without any rise in t
is at a lower temperature than 32° F., or if the water res
rises above 32° F., additional heat will be absorbed as s

The Specific Heat of a substance is the ratio of the q
to raise the temperature of a certain weight of the s
that required to raise the same weight of water from 6

The Heat of Vaporization of water or of any other
heat it will absorb in vaporizing, in evaporating from a
give out in returning from the gaseous to the liquid sta

Transfer of Heat occurs in three ways: (1) by conv
and (3) by conduction. For instance, if particles of
jacent to a source of heat become warmed they circulate
by CONVECTION through the refrigerator-box. Heat will
stance, as from the filament of an incandescent lamp
RADIATION. Heat will enter the box through the walls

Heat-Transmission. When the temperatures on op
face, as for instance, a wall, are unequal, heat will pass
the material from the warmer to the cooler side. The r
the RATE OF HEAT-TRANSMISSION and is stated in terms
called (Btu) which will pass through 1 sq ft of surface
temperature-difference between the two sides of the wall

Some Advantages Claimed for Mechanical Ref

(1) Lower temperatures can be obtained with refri
with ice.
(2) The inconvenience of handling ice is avoided.
(3) There is no accumulation of slime in the ref
melting of even the best ice.
(4) Refrigerators cooled mechanically are dryer

on of Refrigerating-Machines. In almost all methods of producing
ntage is taken of the fact that when a liquid evaporates it usually cools
and its surroundings, and changes into a gas or vapor. There are
uids which are easily made to evaporate and produce this cooling
were it not for their cost, refrigeration could be very simply produced
ng a steady stream of the liquid and allowing the vapor or gas evapo-
scape into the atmosphere. A refrigerating-machine is practically an
for saving this gas which has evaporated and returning it to its liquid
used over again. In this process of recovery and condensation the
out the heat which it has previously absorbed in evaporating. This
rried away by flowing water, which, in absorbing the heat, rises in
re.

of Refrigerating-Machines. In the (1) COMPRESSION-TYPE of re-
-machines the recovery of the gas is effected by drawing it away from
where it has been evaporated and pumping it under increased pressure
mber where it gives out its heat to the water-cooled walls of the
nd returns to the liquid state ready to be used over again. In the
PTION-TYPE of refrigerating-machines ammonia is generally used and
ry of the gas is effected by bringing it into contact with water with
nites chemically. The solution thus formed is pumped into another
nd heat is applied to drive off the ammonia-gas which is then condensed
pressure. It is now ready to be reevaporated and reproduce its cool-
In all cases of large units, and in all cases of either large or small
e exhaust-steam is available in sufficient quantities, absorption re-
-machines are very economical.

Used in Refrigerating-Machines. A number of liquids have been
frigerating-machines, the ones commonly employed being (1) AM-
) CARBON DIOXIDE and (3) SULPHUR DIOXIDE. Various practical
ions determine which is to be used in any particular design of machine.
AMMONIA the advantage is the lower working pressures, from 15 to
sq in, which are easy to deal with. An advantage over carbon dioxide
ks are very easily located. Ammonia-fumes, however, are offensive
imes dangerous in case of a break. With (2) CARBON DIOXIDE the
is in its inoffensive odor. Its disadvantages are the high pressure at
orks, from 300 to 1 200 lb per sq in, the relative difficulty of holding
ures and of finding small leaks, owing to its slight odor and chemical
With (3) SULPHUR DIOXIDE the advantage is its comparatively low
essure, which is not above 75 lb per sq in. Its great disadvantage is
moisture it forms an acid which rapidly corrodes the apparatus. At
his disadvantage was fatal, since with the old-type machines, air and
ere constantly being drawn into the system more or less rapidly and
the sulphur dioxide. This difficulty has recently been overcome in
rn types of machines * in which the refrigerant is hermetically sealed
hine and chemical action, therefore, prevented.

f Refrigerating-Machines. A 1-TON REFRIGERATING-MACHINE is a
hich, if operated for 24 hours, will absorb the amount of heat which
would absorb in melting. If the machine is operated a shorter time
less amount of heat will of course be absorbed, and in order to main-
mperature during the period when the machine is not running, some

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idiffren Refrigerating-Machine, *a small machine intended for domestic uses*
the H. W. Johns-Manville Company, New York. There are many other
making refrigerating-machines of other distinct types, and the architect
refully into the merits and claims of each when called upon to specify them.

(1) In the DIRECT-EXPANSION SYSTEM the refrigera
of pipe placed directly in the room to be cooled.

(2) In the BRINE-SYSTEM the refrigerant is used to
circulated through coils of pipe in the room to be cool

(3) In the COLD-AIR SYSTEM a current of air is chill
of pipe cooled directly by the evaporating refrigera
passing it through a spray of cold brine; and this chill
the room and circulated back to the cooling-coils, th
repeated indefinitely.

All of these systems have their advantages and di
brine-system is a little more expensive to operate in
ature is more easily controlled than with the direct-
practice in small plants it is found as economical in
theoretical disadvantage. Furthermore, in case of an
chine, the temperature can be held for a time by circ
becomes too warm to be of use, whereas with direct e
will begin to rise immediately upon the stopping of th
system is not as applicable where any drying of the goo
ful and there is some risk of carrying fire in the air-pa
nevertheless, for such service as chocolate-dipping roo
fur-storages, etc.

Storage of Cold. When temperatures are to be ma
erating-machine is shut down, COLD must be STORED.
is effected by cooling a comparatively large body of b
as it is circulated. Where the brine-circulating pum
must be stopped, so-called PRESSURE-TANKS may b
system in the room being cooled; the mass of brine in
heat and helps to maintain an approximately even t
direct-expansion system is used, a part of the cooling-
a tank of brine placed in the room and the remainde

ıy be obtained in the refrigerator, since the brine must be ıe ice in order to melt it, and the refrigerator just that much ner than an ice-cooled box. In calculating the proper sizes of brine, it should be remembered that, usually, the period during ine is shut down coincides with the period during which the igeration in the box is the least. The amount of heat to be ılly only that entering through the insulation, as the doors are l is put in or removed.

of Refrigerating-Machines. As explained in the preceding igerating-machines may be divided generally into two classes, SION-TYPE and (2) the ABSORPTION-TYPE.

ression-Type of Refrigerating-Machines may be subdivided as

type of machine, which is made both vertical and horizontal, and double-acting, that is, compressing the gas at one end or the cylinder. (*b*) The partially enclosed type of machine, in ving parts of the compressor proper are enclosed within the frame or, except the fly-wheel and the main shaft which enters the chine through a stuffing-box. Such valves, also, as are required e exposed. (*c*) The wholly enclosed type of machine,[*] in which g parts are enclosed in a hermetically sealed container.

ıtage of the open type of machine is that any lack of adjustment be readily corrected; so that, with proper attention, it gives For large installations this is considered by many to be a pe of machine.

sed type of machine resulted from the effort to reduce the amount uired by the open machine, to cheapen its construction and to ibility of trouble from inexpert tampering. An objection to type is that when adjustments have to be made the working ely inaccessible.

wholly enclosed type of machine it is claimed that the loss of prevented by the hermetical sealing of the apparatus, and that ts, being completely enclosed, are protected from deterioration auses or tampering.

rption-Type of Refrigerating-Machines are of two kinds, differ- the proportioning of the parts. In the one machine high-pres- sed; in the other the proportions are such that low-pressure n may be used. Where exhaust-steam is available machines found to be very economical, and this is true, also, for all large not exhaust-steam is used. Full descriptions of these machines ns and layouts may be obtained from the various manufacturers.

for the Capacity of a Refrigerating-Machine. Heat enters compartments, (1) through the walls, (2) with warm goods, (3) ıge of the outside air when doors are opened and by air-leaks, l air is the heavier and immediately flows out when a door is n lights or from the heat of the bodies of workers, and (5) e of state occurring in the goods, such as freezing, fermenting, ooms these various sources of heat should be analyzed sepa- ll refrigerators, as in hotels, kitchens, dwellings, etc., a rough curate as a more *elaborate analysis, allows a certain* number of *oot of refrigerated space per 24 hours.* This amount varies

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[*] *Referred to on page 1605.*

with the character and location of the box, the nature of its
peratures desired and so on. It will be seen that the insula
importance, is not by any means the only important fac
boxes. For domestic refrigerators in which a temperature o
is maintained, 300 Btu per cu ft of refrigerator per 24 hours
For boxes in hotel or restaurant-kitchens, 600 Btu, or e
treme cases and where low temperatures are required, shoul
butchers' coolers or large storage-boxes in hotels, etc., from
cu ft per 24 hours should be allowed. A check on the ab
large type of box is the following: * "When the exact con
cold-storage rooms are to be operated are known, namely, t
the rooms, the quality of the insulation, the kind and quant
handled per day and the temperatures at which they are rec
they are to be held, the amount of refrigeration required can
closely by the following rule: (1) Calculate the exact area
in the walls, floor and ceiling of the room in square feet, mult
ber of square feet by the number given in the table for the
ture and divide the product by 288 000. (2) Multiply the
in pounds, to be stored per day by the number of degrees of h
by the specific heat of the goods, and divide by 288 000.
amount of refrigeration, in tons per day, necessary to mai
ture required for the goods. (3) Add these two amounts t
will be the amount of refrigeration, in tons per day, requir
temperature required for the goods and for the room. (4
to be frozen, the latent heat of freezing should be added
Btu to be extracted."

For rooms containing less than 1 000 cu ft

If maintained at 0° F. multiply the exposed surface
If maintained at 5° F. multiply the exposed surface
If maintained at 10° F. multiply the exposed surface
If maintained at 20° F. multiply the exposed surface
If maintained at 32° F. multiply the exposed surface
If maintained at 36° F. multiply the exposed surface

For rooms containing from 1 000 to 10 000 cu

If maintained at 0° F. multiply the exposed surface
If maintained at 5° F. multiply the exposed surface
If maintained at 10° F. multiply the exposed surface
If maintained at 20° F. multiply the exposed surface
If maintained at 32° F. multiply the exposed surface
If maintained at 36° F. multiply the exposed surface

For rooms containing more than 10 000 cu ft

If maintained at 0° F. multiply the exposed surface
If maintained at 5° F. multiply the exposed surface
If maintained at 10° F. multiply the exposed surface
If maintained at 20° F. multiply the exposed surface
If maintained at 32° F. multiply the exposed surface
If maintained at 36° F. multiply the exposed surface

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* Taken from Levey's Refrigeration Memoranda, pa

ıchines it is necessary to allow a greater capacity of machine f box than with large machines, since, with the latter, one can large part of the machine-capacity to any given box where exist; whereas to do this with the small machine would almost ıe other box, if indeed there happened to be another box. It is o determine with mathematical certainty exactly how much equired for a given case. It is best to allow for this fact and achine is amply large. Where an existing ice-cooled box is to nically one check upon the size of the machine required is the ed. This check is more apt than any other, however, to lead clusions unless the figures are properly analyzed.

od of Determining the Capacity of a Refrigerating-Machine. a method that gives good results, except that allowance may be er boxes and where brine-storage tanks are provided in the box effect of the mass of cold brine:

nsumption for the hottest month of the year should be deter- ll give the average ice-consumption for that month.

ge temperature that is maintained in the box with ice should ly determined. This will usually be from 55 to 65° F. It will ted to be anywhere from 40 to 45° F., but these temperatures ned. Even if they are, with a full ice-chamber and the box eriods the average will be above these figures. Unless, there- sitive assurance to the contrary, from 55 to 60° F. should be verage temperatures.

ion should then be made of the heat-inflow through the insula- ıperature of 55° F. in the box and with the average summer side. The difference between the heat-inflow through the ıe total heat actually absorbed by the melting of the ice is the the box from other sources than through the insulation. This dinarily occurs during the hours of daytime only, that is, when opened, since at night the box will remain closed. A machine acity to produce the temperature actually obtained with ice be of larger rated capacity than that indicated by the actual and how much larger it should be can be determined by this

fact which it is claimed should be taken into account in deter- er size of a machine is that temperatures obtainable with ice factory. If they were always satisfactory one reason for put- ıachinery would be done away with. Where 55° F. is obtained 5 to 45° F. will be required with mechanical cooling and the st be further increased in the ratio of the temperature-differ- verage summer temperatures and 35° F., and average summer 1 55° F.

g-machine if installed in accordance with these figures would weather conditions but would not be adequate for extreme litions, the most important conditions to be met by cooling- necessary, therefore, to further increase the size of the machine he difference in temperature between maximum summer tem- F., and average summer temperature and 35° F.

allowance should be considered, namely, the fact that in many son or another, it is not possible, or else not desirable, to oper- except during certain periods of the day, and the machine-size 1 as much as may be *required to take care of these* conditions.

(7) If the machine is not placed directly at the box to be cooled, allo must be made for the heat-inflow into the insulated brine-mains. The of heat entering from this source is often of considerable importance, larly with small machines. The table below gives heat transmissions pipe-covering and some other materials.

Water and Milk-Cooling. Mechanical refrigeration as applied to water and milk differs in one respect from other classes of refrigerating A relatively intense quantity of cooling effect is called for in a brief of time. For instance, in a drinking-water system the heaviest require may come at the noon-hour. In a bakery, also, the demand for chilled will be intermittent, a large quantity of water being required for the mixing. In dairy-work the milk must be cooled very rapidly to che development of bacteria which grow with incredible rapidity within the perature-range of from 110 to 50° F. To install a large enough refrig machine to produce the required cooling effect as it is needed would in cases call for a very large machine. This is overcome by using a machine and allowing it to operate for a longer time, say through day, storing the refrigerating effect produced by cooling a large body of or melting the ice as rapidly as may be required. For instance, if 50 of milk, of 40 qts each, are to be cooled from a temperature of from to 35° F., in 1 hour, the refrigeration required will be 50 cans times 40 qts 2 lb per qt times (75° F. − 35° F.), which equals 320 000 Btu. Milk is in the calculation as having the same specific heat as water, since water so large a percentage of its total weight. This amount of refrigeratio duced by a machine running 12 hours per day would require the mach absorb 320 000 Btu divided by 12, or 26 000 Btu per hour. The quant brine necessary to store the cooling effect may be calculated closely practical purposes by using the following approximate figures. The heat of brine is 0.75. The weight of the brine is 9 lb per gallon. The sible temperature-range of the brine depends upon the conditions and from, say, 30 to 15° F., or lower. In other words, the temperature the brine can be permitted to rise is limited to the temperature it must in the room or in the substance being cooled, and the temperature the brine can be cooled in storing cold is limited by the decrease in econ the refrigerating-machine at the low temperatures.

The Value of Good Insulation. (See, also, Insulation, page 160 importance of good insulation cannot be too strongly emphasized. A storage room or refrigerator and its contents may be cooled by ice or cal means, but unless the walls are adequately insulated, the demand by the inflow of heat through the poor insulation may be more than the supply or refrigerating-machine can meet to maintain the required ture. The almost universal standard of insulation for cold-storage 4-in thickness of pure-cork sheet. The following table shows the heat mitted through 1 in in thickness of each of the substances, per square exposed surface per degree difference in temperature per 24 hours.

Pure-cork sheets	6.4 Btu
Hair-felt	7.3 Btu
Impregnated cork boards	8.5 Btu
Rock-wool blocks	8.0 Btu
Waterproofing lith-blocks	8.5 Btu
Spruce, clear and dry	16.0 Btu
White oak	26.0 Btu

f Refrigerators. Disposition of Cooling-Surfaces. (See, also, frigerators, page 1599.) No attempt need be made to describe all rrangements of refrigerated compartments that are to be found in intention is to point out some of the more important things to l in determining upon the design of a box. It is desirable in a o produce not only a low temperature, but a relatively dry atmos- [illegible]

rface and Temperature. Securing the low temperature is merely supplying sufficient cooling-surface to produce the desired results perature available in the refrigerant. The amount of surface fluenced by the arrangement of the box, that is, whether or not freely or sluggishly over the surface, whether the cooling-surface he ceiling or walls of the compartment or in a loft and, if the latter is used, whether or not the air-passages are of proper size and the tween the loft and the compartment sufficient.

Atmosphere and Temperature. To secure a box of satisfactory necessary to have a relatively low temperature in the refrigerant. passes over the cooling-surfaces is practically in a saturated con- t leaves them. If it is to be dry at the temperature required in ust have been, necessarily, cooled well below the box-temperature. in a box, the temperature of which is maintained at 35° F., the be run at a temperature of from about 20° to 25° F. It is further o locate the cooling-surface that frost in melting will pass out of tly and not remain to be reabsorbed by the air in the box.

nts of Cooling-Surfaces. There are several common arrangements faces in refrigerators. Sometimes the coils are arranged overhead, n the compartment to be cooled. This is one of the efficient ways oling-surface can be arranged, so far as the cooling effect alone is It is not, in general, a good arrangement, however, since frost the coils drips on the goods. In another arrangement the cooling- on the wall. This is preferable to the ceiling-arrangement, as far ng is concerned. The objection to it is that goods placed close to apt to be overchilled, while goods nearer the center of the com- not cooled quickly enough. It also wastes floor-space, because s close to the coils is not practicable on account of possible over- lso on account of the liability of retarding the air-circulation. The nent for cooling-surfaces is, nevertheless, often the most practica- Another method involves a modified form of wall-coil arrangement ine-storage tank is used to assist in maintaining the temperature chine is shut down. A further modification is often introduced, rtition or baffle-plate is used in front of the coils. The best types ement are those in which the cooling-surface is separated from the and is so arranged as to secure an active circulation of the air s and through the compartments. In all of these plans the one calling for the greatest care is that the air-passages shall be as sible and of ample size. The force causing the air to circulate, difference in weight due to differences in temperature and density column of air in the coil-compartment and that in the storage- , is so extremely small that any slight interference is a serious extra turn in the passage or a slight reduction in the size of the produce a marked effect. A good rule to follow is to make the rge as it can be made without allowing any drip to reach the rtment. This will work out in many cases to show a ratio of

faces well away from walls or ceilings, from 3 to 8 in
perature of the brine.

Incidental Notes on Refrigerators. **Drawers**
and elsewhere it is sometimes convenient to have a
of refrigerated drawers. The heat-leakage through
slides which are invariably only partially closed, an
tion of the drawers, is very great. Where it is at
best to arrange an insulated door covering the entire

Anterooms. In storage-rooms of medium to lar
due to opening doors is reduced to a minimum by
entry which, after it is entered, has its outer door cl
storage-room proper is opened. Where two rooms a
possible to reduce the interchange of air by treating
room of the other, having but one door to the outsid

Doors. Special note should be made as to the desi
rooms or boxes. There is a common idea that a r
beveled. As a matter of fact no more certain mea
could be devised. A perfectly fitted beveled door,
could perhaps be made tight in the beginning. Th
begins to sag, since a refrigerator-door is always he
comes impossible to force it to a tight seat and contin
A refrigerator-compartment door is most readily ma
surface on the door come up against a corresponding
a soft gasket of some kind between them. There
frigerator-doors on the market at prices low enough to
to attempt the home-made article.

Arrangement of Brine-Mains. In laying out main
refrigerating-machine to the refrigerator, there are a
cared for. For the convenience of the pipe-covering
lines should be placed far enough apart so that he
each pipe without cutting it to pieces, or else they s

ins is as nearly as possible equal to the combined cross-section ils which they serve at any one time. Even with this proportion, not possible to absolutely ensure that the lower coils will not rob s, or even drain them completely in some systems of piping. A e, even if somewhat expensive method of overcoming this diffi- e addition of a third main. In this arrangement it is not possible rob another to the point of draining it.

ns for the Necessary Amount of Cooling-Surfaces. No rule can be given regarding the proper amount of cooling-surface ents of various sizes, since the design and arrangement of the e and the freedom with which the air circulates over it greatly ount required. As a general guide, however, and where the con- ch as to permit a good circulation of the air, the following formula l results. It will be understood, of course, that the refrigeration the given room has been determined as previously indicated. urface required, in square feet, per ton of refrigeration equals in which T is the temperature desired in the compartment, and t emperature of the brine.

Approved Cold-Storage Temperatures

Articles stored	Degrees Fahrenheit
	36 to 40
utton	32 to 36
	29 to 32
	34 to 36
kle or brine	35 to 40
be kept separate from other goods	0 to 38
	29 to 32
	32 to 34
	38 to 40
reeze	5 to 10
n frozen	25 to 28
eze	5 to 10
frozen	25 to 28
fish-counters should be cooled with ice rather than	
lly	25 to 28
	33 to 45
	33 to 42
	40 to 45
	30 to 40
	33 to 36
	34 to 40
ls	38 to 40
eal	40
	25 to 32
cream freezing	5 to 10
r-hardening	5
rving-temperature	14 to 16

g. If the following facts of physics are kept in mind in consider- of making ice the results obtainable may be understood or pre-

collect near the surface, and the heavier or
(6) The rate of freezing of ice decreases as th
increases, so that the time required to fr
of the thickness to be frozen. In the fo
freezing is from the top down and impuriti
This and the motion of the water, especial
tends to make naturally frozen ice transp
turers of ice have always tried to duplicat

Methods of Ice-Making. The method first ado
one in which DISTILLED WATER was used. From a
ice would be theoretically ideal. Practical difficu
sible to secure pure ice in this way. Some of thes

(1) Removal of oil from the distilled water, thi
steam passes through the cylinder of th
remove organic oil which is present in the
(2) Assurance that the filters are in proper shap
sible to obtain since this apparatus is ordin
without overhauling.
(3) Possibility of contamination in the storage-t
is held and usually PRECOOLED to as ne
passing to the freezing-cans, thus saving
in the tank.
(4) Possible contamination from handling the
over them. These covers form the top o
the cans of water are immersed in cold
tramped over by the ice-harvester with t
dirt getting into the cans.

A second system of ice-making in common use
SYSTEM. In this process the ice is formed on ve
or raw water is used and the bath is agitated by
sulting ice is very clear and dense. In this syste

gine would use and it was not possible to obtain fuel-economy. form of this system, now coming into considerable favor, is at STATIONARY CANS are filled with raw water and kept agitated air bubbling up through it. When the freezing has progressed remaining water is drawn off and replaced by fresh water, thus greater part of the impurities that have been frozen out of solu- other modifications of these two systems of ice-making have been developed. All of them depend, however, upon the series of stated in the preceding paragraphs, and the results may be reference to them.

conomy of Producing Refrigeration Mechanically and by determining the cost of REFRIGERATION BY ICE, account must only of the cost of the ice but of melting, of the uncertain ice- e amount of ice left over at the end of the season and of that er in the storage and, therefore, practically useless. Regarding t may run anywhere up to 50% of the total ice-harvest. The over at the end of the season is, of course, so variable that it is estimate it, this being purely a matter of chance. In many r, it is a very large item. The loss by the ice freezing together can be reduced to a very small amount where the ice is properly distance-strips between the ice-cakes. Proper packing is much carried out, however, where artificial ice is stored than where held, and a mechanically cooled ice-storage is less subject to this e the temperature is, of course, constantly held below the melting- (2) The total cost of REFRIGERATION PRODUCED MECHANICALLY cost of power, water, oil, refrigerant (usually ammonia), labor e, and interest and depreciation on the investment. The figures s vary between wide limits. The following figures, however, will . Care should be taken in drawing conclusions from them as to ctive installations. These figures are from the annual cost of an ring company having a capacity of 1 500 tons per day in plants from 50 to 100 tons per day each.

....	40 cts per ton of ice produced.
....	50 cts per ton of ice produced.
....	10 cts per ton of ice produced.
....	5 cts per ton of ice produced.
ower, oil, etc.............	10 cts per ton of ice produced.
tal.....................	$1.15 per ton of ice produced.

TOWER-CLOCKS *

iameter of Dials. "To look well and show plainly, dials should meter for every 10 ft of elevation and should set out flush with or ne of the building or tower." †

s of Some Large Clock-Faces. Colgate's Factory, Jersey The diameter of the dial is 40 ft. The minute-hand is 20 ft long in extreme width, and the hour-hand is 15 ft long and 3 ft 10 in idth. The minute-hand weighs 640 lb and the hour-hand 500 lb. rgest clock in the world.

ription of the requirements of installation of tower-clocks, see page 154 nd Chapels," by F. E. Kidder.

as Clock Company, New York.

Bromo-Seltzer Building, Baltimore, Md. The dials are 24 ft in diamet The minute-hand is 12 ft 7 in and the hour-hand 9 ft 8 in from tip to tip. The minute-hand weighs 175 lb, the hour-hand 145 lb.

Daniels-Fisher Building, Denver, Colo. The dials are 15 ft 6 in in diamete The minute-hand is 7 ft 10 in and the hour-hand 5 ft 7 in long.

Maryland Casualty Building, Baltimore, Md. The dials are 17 ft in diam- eter. The minute-hand is 8 ft 4 in and the hour-hand 5 ft 11 in long.

Elgin Watch Company's Factory, Elgin, Ill. The dials are 14 ft 6 in in diam- eter. The minute-hand is 7 ft 4 in and the hour-hand 5 ft 4 in long.

Tower-clock, Station of the Central Railroad of New Jersey, at Commu- paw, N. J. The diameter of the single dial is 14 ft 3 in; the minute-hand i 7 ft long and weighs 40 lb; the hour-hand is 5 ft long and weighs 28 lb. The motive power is furnished by a weight of 700 lb, hung from a ¾-in steel cabl

Four-dial clock, Produce Exchange Building, New York. The diameter of each dial is 12 ft 6 in.

Four-dial clock, Chronicle Tower,* San Francisco, Cal. The diameter of eac dial is 16 ft 6 in; length of minute-hands, 8 ft; length of hour-hands, 5 ft 6 in The mechanism of the clock is 6 ft 1 in high and weighs 3 000 lb.

Pneumatic clock, City Hall and Court-House, Minneapolis, Minn. The dial are 23 ft 4 in in diameter.

LIBRARY BOOK-STACKS

The Stack-Work in General. The stack-room of a library is usually c off by fire-proof doors from the rest of the building. The customary practi among architects is to make the stack-work a separate contract and have th general contractor turn the stack-room over to the stack-contractor with fi ished floors, walls and ceilings. The stacks, made entirely of incombustib materials, are then built as an independent structure.

Book-Ranges. The book-ranges are usually double-faced and are plac in parallel rows with aisles between. The minimum aisle-width is about 2 4 in. Radial ranges waste space and are costly. Single-faced ranges a relatively more expensive than double-faced ranges.

Tiers. All stacks are divided in their height into tiers by deck-floors order that all shelves may be easily reached. The regular tier-height is 7 or 7 ft 6 in.

Deck-Floors. Deck-floors are composed of slabs of ¾-in rough pl glass or 1¼-in white marble, supported on steel framework. A long, narr opening or deck-slit is left between the edge of each deck-floor and the face each range to allow proper ventilation of the stack-tiers. The net thickn from top of deck to bottom of steel framework is from 3¼ to 3¾ in for ordina spans. The deck-floors are carried by the shelf-supports.

Vertical Communication. Continuous flights of stairs of simple des and construction are placed at central points. Books are moved up and do by means of dumb-waiters operated by hand, for short runs, or by elect power controlled by push-buttons.

Shelf-Supports. The shelf-supports are made in various ways, differi with each manufacturer. In the best construction they extend the full wid of the shelves so as to hold up the shelves and books without the use of a projecting brackets. They are made of sufficient strength to carry the co bined loads of books, deck-floors and superimposed stack-tiers. They sho

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* Destroyed in the earthquake and fire.

r a uniform shelf-adjustment at intervals of about 1 in. Compact-
portant. Open-work shelf-supports promote proper lighting and

. In each tier of regular height there are usually six rows of adjust-
s and one row of fixed shelves. Shelves are generally 8 or 10 in wide
ng. Other sizes are supplied if necessary. The adjustable shelves
f solid plates of sheet steel or of parallel bars with spaces between.
helves are placed about 2 in above each floor-level. They are made
tes of steel to form dust-stops, fire-stops and water-stops between the

The adjustable shelves are always completely finished with baked
ore delivery. The fixed parts, also, of the stack-construction may
at the shop with baked enamel, or preferably with air-drying enamel,
on at the building, so as to permit repair.

. Electric-light wires are carried in metal conduits supported by
amework of the deck-floors. Lights of 16 candle-power are spaced
apart in range-aisles and 12 ft apart in main aisles.

. Indirect radiation is best for books. The lower tiers, only, of a
d be heated, to prevent the upper tiers from becoming too warm.

ion. Large stacks are usually ventilated artificially to prevent the
st and outside air through open windows. In the Library of Con-
ashington, D.C., fresh, filtered and tempered air is forced in at the
r, finds its way up through the stack by means of the deck-slits and
ut at the top tier.

. The shelves and shelf-supports * weigh from 7 to 10 lb per cu ft
ge. Books weigh about 20 or 25 lb per cu ft of book-range. The
floor framing weighs from 4 to 6 lb per sq ft of gross area of deck-
rble floor-slabs, 1¼ in thick, weigh about 20 lb per sq ft, and ¾-in
e-glass slabs, about 10 lb per sq ft of net area.

pacities. Book-capacities per linear foot of shelf may be figured on
g basis: law-books, 5 volumes; reference books, 6 volumes; scien-
7 volumes; general literature, from 8 to 10 volumes. The average
ary of Congress is 8½ volumes per linear foot. An ordinary stack-
ves high with double-faced ranges 16 in deep (or 8-in shelves) and
wide, with a reasonable allowance made for cross-aisles, stairways,
ntain about 22 volumes per sq ft of gross area.

he cost in the United States of library-stacks of standard construc-
from 50 cts to $1 or more per linear foot of shelving. Economy is
following established standards while special designs increase the cost.

CLASSICAL MOLDINGS

s are so called because they are of the same shape throughout their
ough the whole had been cast in the same mold or form. The regu-
s, as found in remains of classic architecture, are eight in number,
n the accompanying illustration, and are known by the following
e last two are commonly called, also, OGEE MOLDINGS. Some of these
erived thus: FILLET, from the French word FIL, a thread; ASTRAGAL,
GALOS, a bone of the heel, or the curvature of the heel; BEAD, because
g, when properly carved, resembles a string of beads; TORUS, or TORE,

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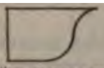
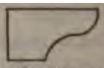
* As made by The Snead & Co. Iron Works, Jersey City, N. J.



Cymatium, or cyma-recta.



Inverted cymatium, or cyma-reversa.

also of the ast
resemble rope
The ovolo and
at their upp
therefore used
parts above them. The cyma-recta and cavetto, bei
extremities, are not used as supporters, but are placed
shelter the upper parts. The scotia is introduced in t
separate the upper and lower torus, and to produce
relief. The form of the bead and that of the torus ar
for giving distinct names to them is that the torus, in
considerably larger than the bead and is placed am
whereas the bead is never placed there, but on the
The torus, also, is seldom carved, whereas the bead is
among the Greeks, was frequently elliptical in its for
circular shape. While the scotia is the reverse of the t
reverse of the ovolo, and the cyma-recta and cyma-re
of the ovolo and cavetto.

THE CLASSICAL ORDER

Origin of the Orders. "In the classical styles sev
and entablature are in use. These are called the ORDERS.
a COLUMN with a BASE, SHAFT and CAPITAL, with or with
BASE, DIE and CAP, and is crowned by an ENTABLATURE, c
FRIEZE and CORNICE. The entablature is generally abou
the column, and the pedestal one-third, more or less.
forms used by the Doric race, which inhabited Greece it
Sicily and Italy, were much unlike those of the Ionic ra
western coast of Asia Minor, and whose art was great
Assyria and Persia. Besides the IONIC and DORIC styl
a third, which employed brackets, called MODILLIONS, i
much more elaborate than either of them; this they

vary much among themselves and differ in different places,
nes still further varieties are found in Italy, Spain, France,
land. The best known and most admired forms for the orders
out by Giacomo Barozzi da Vignola in the sixteenth century
ancient

Order.
ishing
ie Tus-
1) is
forms
in and
show
s, and
con-
CAN."

Order.
ishing
f the
re fea-
ZE and
above
us and
re sup-
ed from
ns and
imitive
iction
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h, and
which
rave, is
led the
er the
drops,
ich are
, some-
There
ighteen
t cylin-
ler the
mutule.
pposed
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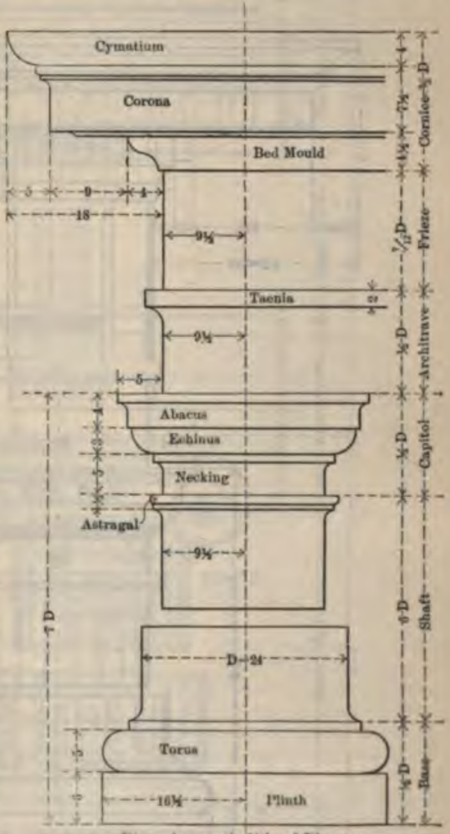


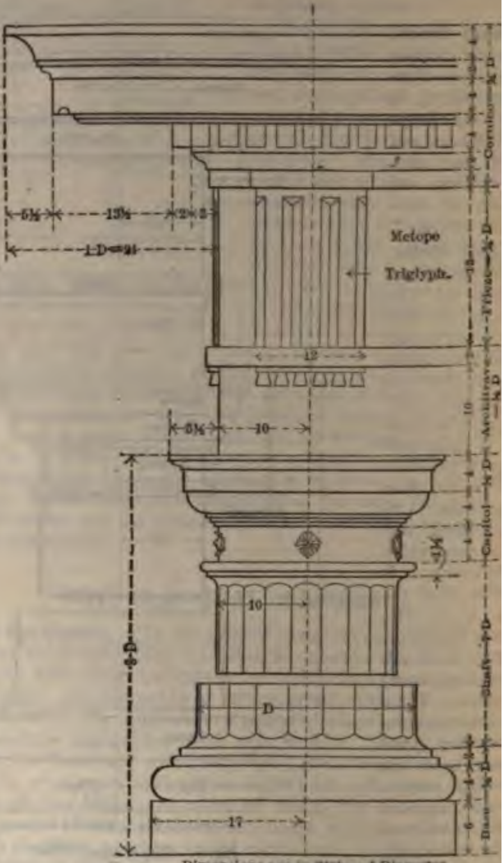
Dimensions are in 24ths of Diameter.

Fig. 1. The Tuscan Order

ie MUTULARY with bracket and the DENTICULATED with dentils,
erence being in the BED-MOLD." The order shown in Fig. 2
ted cornice.

ler. "The prototypes of the IONIC ORDER (Fig. 3) are to be
ssyria, and Asia Minor. It is characterized by BANDS in the
NTILS in the bed-mold, both of which are held to represent
ogether to form a beam or a roof. But the most conspicuous

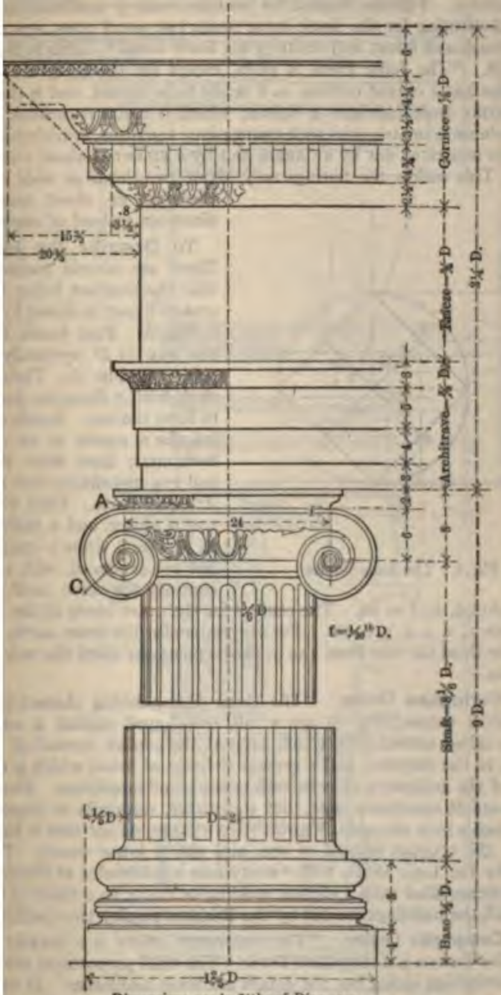
and distinctive feature is the SCROLLS which decorate the CAPITAL of
These have no structural significance, and are purely decorative
from Assyria and Egypt. Originally the Ionic order had no
ECHINUS in the capital. These were borrowed from the Doric



Dimensions are in 24ths of Diameter

Fig. 2. The Doric Order

like manner, the dentils and bands in the Doric were borrowed from
The Ionic frieze was introduced in order to afford a place for sculpture
called by the Greeks the ZOOPHOROUS, or figure-bearer. The typical
is considered to consist mainly of a SCOTIA, as in some Greek
common, however, to use instead what is called the ATTIC BASE,



Dimensions are in 24ths of Diameter.

Fig. 3. The Ionic Order

diameter,

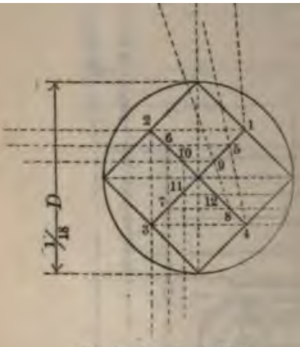
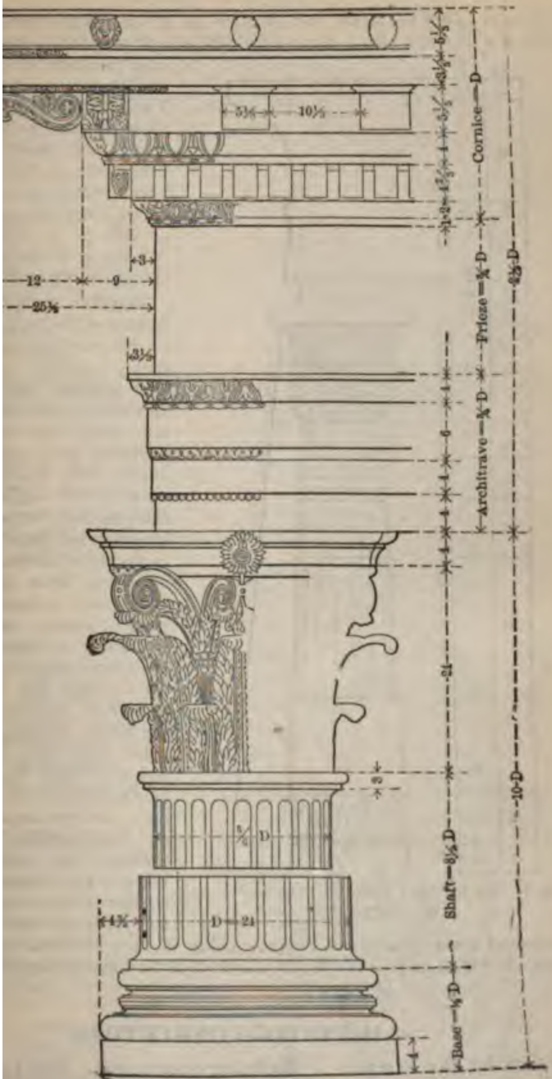


Fig. 4. The Ionic Volute

To De
There ar
this, the
centers fo
in Fig. 4.
the EYE
point A,
circle with
to form th
inscribe a
horizontal
and 2–4,
six equal
1 as a cen
to A, Fig.
line 1–2 p
continue

2–3 produced, and so on. The centers for the out
the points 1, 2, 3, 4, 5, 6, etc. For the centers for
point one-third the way from 1 to 5, then a point on
and so on.

The Corinthian Order. "The three distingui
CORINTHIAN ORDER (Fig. 5) are a tall, bell-shape
brackets called MODILLIONS, which support the cor
addition to the DENTILS, and a general richness of
the use of the ACANTHUS LEAF in both capitals and
ATTIC BASE is commonly used, but sometimes, es
base is used which resembles Vignola's IONIC BASE,
between the SCOTIAS instead of one, and also a
fluted like the Ionic shaft, with twenty-four semic
are sometimes filled with a convex molding or CABI
Almost all the buildings erected by the Romans



Dimensions are in 24ths of Diameter

THE CORINTHIAN ORDER

Fig. 1. The *Corinthian Order*

DENTILS. They are larger, and are more nearly square in elevatio fifth of a diameter high and one-sixth wide, the INTERDENTIL bein

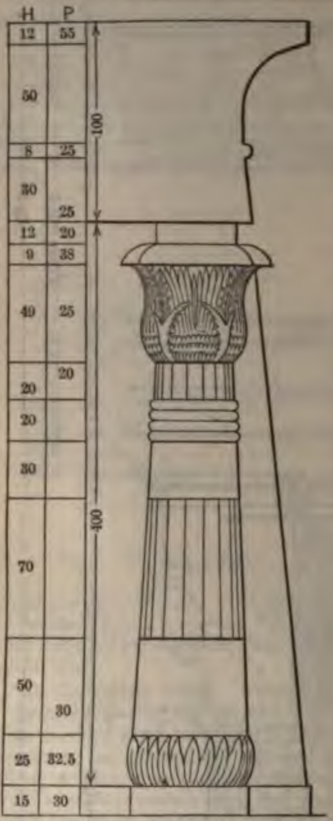


Fig. 6. An Egyptian Order. Diameter Divided into Sixty Parts

and they are set a diameter apart, The composite c ployed in the Arc Rome, and elsew Corinthian entabla BLOCK CORNICE occ called FRONTISPIE as well as in th Athens, in connec Corinthian capital.

Egyptian Style. tecture of the anci is characterized b outline, solidity, The principal fea EGYPTIAN STYLE o are: uniformity o deviating from ri angles; thick wall outer surface sligh inwardly from the the whole building composed of stone one piece from these being suppo mous columns, v proportion to thei shaft sometimes p ing no base, but variety of handso the foliage of thes palm, lotus and ENTABLATURES h an ARCHITRAVE, cr huge CAVETTO orn sculpture; and the NIATION very na 1½ diameters and ceeding 2½. A g larity exists in the forms and general features of Egyptian columns. For practical us shown in Fig. 6 may be taken as a standard of the Egyptian style

LIGHTNING-CONDUCTORS

Rules for the Erection of Lightning-Conductors. The fo for the erection of lightning-conductors were issued in 1882 by the D Explosives of the English Home Office to the occupiers of all factori

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* From The American House Carpenter, by R. G. Hatfield.

:xplosives, and to those local and police authorities upon whom de-
inspection of stores of explosives:

erial of Rod. Copper, weighing not less than 6 oz per ft run, the
:onductivity of which is not less than 90% of that of pure copper,
he form of rod, tape, or rope of stout wires, no individual wire being
No. 12, Birmingham Wire-Gauge (0.109 in) the English standard
:. Iron may be used, but should not weigh less than 2½ lb per foot

ts. Every joint, besides being well cleaned and screwed, scarfed,
, should be thoroughly soldered.

n of Points. The point of the upper terminal* of the conductor
: have an angle sharper than 90°. A foot below the extreme point a
g should be screwed and soldered on to the upper terminal, in which
d be fixed three or four sharp copper points, each about 6 in long.
ıble that these points should be so platinized, gilded, or nickel-plated
t oxidation.

ıber and Height of Upper Terminals. The number of conductors or
ninals required will depend upon the size of the building, the material
it is constructed, and the comparative height above ground of the
ırts. No general rule can be given for this, except that it may be
:hat the space protected by the conductor is, as a rule, a cone, the
whose base is equal to the height of the conductor from the ground.

vature. The rod should not be bent abruptly around sharp corners.
: should the length of a curve be more than half as long again as its
. hole should be drilled in string-courses or other projecting masonry,
ible, to allow the rod to pass freely through it.

ılators. The conductor should not be kept from the building by
:her insulators, but attached to it by fastenings of the same metal as
e conductor itself.

ng. Conductors should preferentially be taken down the side of
ng which is most exposed to rain. They should be held firmly, but
ısts should not be driven in so tightly as to pinch the conductor or
ɔntraction and expansion due to change of temperature.

er Metalwork. All metallic spouts, gutters, iron doors, and other
metal about the building should be electrically connected with the
.

th-Connection. It is most desirable that, whenever possible, the
:emity of the conductor should be buried in permanently damp soil.
oximity to rain-water pipes and to drains or other water is desirable.
:y good plan to bifurcate the conductor close below the surface of the
nd to adopt two of the following methods for securing the escape of
ing into the earth: (*a*) A strip of copper tape may be led from the
the rod to a gas or water-main (not merely to a leaden pipe), if such
enough, and be soldered to it; (*b*) a tape may be soldered to a sheet
, 3 by 3 ft by 1/16 in thick, buried in permanently wet earth and sur-
ɔy cinders or coke; (*c*) many yards of copper tape may be laid in a
ed with coke, having not less than 18 sq ft of copper exposed.

otection from Theft, etc. In places where there is any likelihood of
ır being stolen or injured, it should be protected by being enclosed

---

pper terminal is that portion of the conductor which is between the top of the
the point of the conductor.

350 ft in height, for the plant of the St. Joseph Lead
Mo. (See article on Radial Block Chimneys, page 12

Conductor. The conductor used is of commercially p
& Sharpe gauge, in the form of a cable, consisting of
strands, four wires to the strand, and ⅝ in in diame
The vertical conductors are of continuous lengths fro
to and into the ground. A circuit-conductor is plac
the chimney and connected to each down-conductor b

Points. The air-terminals are eight in number eq
top of the chimney, and consist of solid, copper bars
length, the upper 12 in tapering to a point and cover
genuine platinum. Air-terminals extend 5 ft above
the lower end of each copper bar is set in a heavy cop
nects the same into the circuit-conductor. Each rod
anchor-fasteners, bolted from the inside of the stack.
cased in copper tubes set in the solid masonry.

Grounding. At a point below the ground-level
the conductor is carried in a downward course from
bedded in charcoal, to a point 5 ft outside the founda
conductor is spliced into the main cable at this
branches terminating 15 ft apart. Two well-holes ar
proximately 20 ft into permanent moisture. The en
electrically soldered into perforated, copper reservoirs
in length, and filled with pea-size charcoal. The eff
give the required amount of surface-contact with the
manent moisture through the charcoal by capillary
conductor is thus grounded in two places instead of

Lead Covering. To preserve the conductor syste
in ozone, in which sulphuric or other acid gases may e
system at the top, and to a point 75 ft below the top
with lead ⅛ in in thickness. Exception is made to th
top of each rod, which requires no lead covering. W

t has been found that in from three to five years there is a de-
bing of the copper, through the action of the sulphuric and
. It has often been necessary to replace points, sections of cable,
ten away from this cause.

HONES. AUTOMATIC TELEPHONES FOR INTERCOMMUNICATING SERVICE

. The interphone system is an application of the telephone for
t is an automatic, intercommunicating system, requiring neither
operator, and being self-contained within the walls of the estab-
ose benefit it has been installed.

. In brief, the advantages of such a system are these: (1) the
f a button gives a person telephone-connection with any desired
the loss of time involved in first calling up a third party; (2) re-
ory or information bureau is made unnecessary through the use
ly inscribed, on the face of the instrument; (3) no maintenance-
ved, and the system, consequently, is as inexpensive to operate
or-bell; (4) the wiring-arrangement is such that the system may
when the original plans for a new building are being drawn up,
ect it does not differ much from a system of electric lights or

Interphones in residences, schools, hospitals, factories, mills,
nd clubs is constantly increasing. The same general features
these types of installations, and in practically every case it is
of the system that especially recommends it for service. The
ally fits in where formerly call-bells, speaking-tubes, messenger
er inadequate methods were the rule. The interphone field of
e establishment whose needs call for from four to thirty-two
ns. When there are more than thirty-two the installation of a
ne-exchange, with a switchboard, is better practice.

terphones. There are several types of interphones for varying
e.
t familiar instrument is a wall-interphone, of the NON-FLUSH
phone is of metal, with connecting buttons, labels, bells, mouth-
receiver, all mounted on its face. This instrument is to be at-
to the wall.
H TYPE resembles the first-mentioned type in every particular
lied in its title. The instrument is mounted into the wall, with
th the rest of the wall-surface. These two instruments are most
tallation in club-hallways, in stores and factories, in residences,
s where wall-telephones would ordinarily be used. Busy offices
employ variations of types (1) and (2) and use a DESK-SET, a
nent taking care of the connecting buttons and labels, or a HAND-

-STAND telephone is of the type often used for local and long-
. Connected with it is a metal box containing the rows of
bels, each label being opposite the button through which is se-
n with the corresponding station. The telephone in this case
esk, and the key-box is conveniently close at hand, either on the
wall.

(1) or (2) with a connecting button for each outsid
room-telephones are all of type (5). With this sys
any time call up any teacher; but a teacher can c
only through the medium of this MASTER-STATION, w
change. The advantages of this arrangement for a
hospital the instruments are usually placed outsid
operating-rooms and wards and in the offices and rec

Wiring and Batteries. All wiring is enclosed in c
from dry cells. The only maintenance-expense conn
system is the occasional renewal of these batteries.

VACUUM-CLEANIN

General Description. Vacuum-cleaners are ap
into use during recent years and which are for the
and dust from rooms of buildings, cars, steamship
carpets, curtains, or other interior fittings. The du
suction and the apparatus consists of an air-pump
the air and the dirt or dust contained in it throug
nozzle is drawn or passed over the surfaces which s
of muslin or other appropriate cloth are used to sep
and dirt which are borne along with the stream of
apparatus this process is assisted by what are called b
to make the heavier particles of dust drop by thei
part of the receptacle placed to receive them. Abou
air was used for the first time in railroad-cars for pur
removal. There were serious objections to this me
as it was found that the jets of compressed air blew o
a way that it was difficult to arrange for their col
principle of suction was consequently introduced to

Types of Vacuum-Cleaners. The machines bel
usually consist of a pump, the motor-power of which

to the street-sewers. A portable tank is used for the soap and um-cleaners of a third type consist of small machines which take ie brooms and dusters or are used in connection with them. They generally used and may be driven by an electric motor, by foot, or iese last-mentioned, smaller, portable cleaners are used for many s than the ordinary cleaning of rooms and furniture.

id Specifications for Vacuum-Cleaning Installations. Com- d specifications for the installation of a vacuum-cleaning plant for iy be obtained from any of the numerous manufacturers making s and taking contracts to put it in place. There are several types nd systems of installation and detailed descriptions would exceed pace in this handbook.

ATERPROOFING FOR FOUNDATIONS *

rproofing of Substructure Work is, comparatively speaking, a h of engineering. It is only within recent years that it has become construct deep basements for buildings. In the past, the more uctures, such as cathedrals, capitols, state-buildings and the like, built upon high ground, and water was prevented from entering of such buildings by means of drainage. Waterproofing, as we was generally unnecessary. With the advent of the so-called sky- ever, requiring large mechanical plants, deep basements became essity, and as these basements are usually carried below ground- id in many instances below tide-level, the question became one of tance. Like almost every detail of a modern building, water- pecialty. Each building presents its own problems, and the safest the solution of these problems to some one expert in the knowledge ng who has made it a special study and knows how best to over- ting difficulties. It may be laid down as an invariable rule that, ons are at all serious, the owner or the general contractor will the long run if he employs the services of an expert waterproofer aterproofing-seal, regardless of the method he wishes to use.

Resistance Versus Waterproofing. In waterproofing large base- actual pressure exists, it is a question for the engineer to decide more economical to attempt to secure an absolute PRESSURE-JOB ROOF JOB in connection with a drainage system. As a general e stated that where a building is generating its own power, it is cal to use a drainage system with an open sump than to construct ar, the cost of pumping being much less than the interest charges a floor-slab sufficiently strong to withstand the pressure.

fing Concrete Foundations. The three following subdivisions , discussing the causes of permeability of concrete, the addition of render it more water-proof, and the treatment of its surfaces to ermeable, embody the conclusions of Committee D-8 of the Amer- or Testing Materials.† This committee, since its organization in ugh laboratory-tests and experiments, together with examinations g construction and after completion, as well as the study of liter- ubject, sought to secure sufficient information to enable it to for-

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tions in general see Chapter II.

e, to the middle of page 1633, is the substance of a preprint of the Report he American Society for Testing Materials at its meeting, June 24–28, meeting it was formally accepted.

(1) The determination of causes of the permeabili
made from mixtures of Portland cement, sand and sto
gate, in proportions of from 1 of cement, 2 of sand
cement, 3 of sand and 6 of stone, and the best m
causes.

(2) The rendering of concrete more water-proof by
tures of cement, sand and stone, other substances wh
filling or repellent action, would tend to make the con

(3) The treatment of exposed surfaces after the con
put in place and hardened more or less, either by p
repellent liquids, making the concrete itself less perm
protective coatings, preventing water from having ac

Considering these several subdivisions separately an
committee arrives at the following conclusions:

(1) **Causes of Permeability of Concrete.** In the la
conditions where properly graded and sized coarse
used, in mixtures ranging from 1 of cement, 2 of sand
ment, 3 of sand and 6 of stone, impermeable concre
duced. Even with sand of poor granulometric comp
rich as 1 of cement, 2 of sand and 4 of stone, permea
ever, found and is a rare occurrence with mixtures o
and 6 of stone. But the fact remains, nevertheless, t
tains in actual construction, permeable concretes bei
mixtures of 1 of cement, 2 of sand and 4 of stone and
where the quantity of the aggregate is increased. This
to:

(a) Defective workmanship, resulting from improp
thorough mixing, separation of the coarse aggregate fr
cement in transporting and placing the mixed concret
insufficient tamping or spading, improper bonding of

(b) The use of imperfectly sized and graded aggreg

(c) The use of excessive water, causing shrinkage-

ucing permeable concrete is aggravated by poor and inefficient
if the added expense of screening and remixing the aggre-
forded, so as to secure proper granulometric composition to
equired to make untreated concretes impermeable, it is seem-
mercial impossibility on large construction to obtain work-
roximating that found in laboratory-work.

'oreign Substances to Cement Before or During Mixture. The
hat in consequence of the conditions outlined above, sub-
to make the concrete more impermeable, either incorporated
added to the concrete during mixing, are often used. This
development and placing on the market of numerous patented
terproofing-compounds, the composition of which is more or
et. While it has been impossible for the committee to test all
terproofing-compounds being placed on the market, it has
icient number of these, as well as the use of certain very finely
occurring or readily obtainable commercial mineral products,
nd sand, colloidal clays, hydrated lime, etc., to form a general
f the different types. The committee finds:

ajority of patented and proprietary integral compounds tested
mediate or permanent effect on the permeability of concrete
these even have an injurious effect on the strength of mortar
hich they are incorporated.

ermanent effect of such integral waterproofing-additions, if
action of organic compounds, is very doubtful.

w of their possible effect, not only upon the early strength,
durability of concrete after considerable periods, no integral
erial should be used unless it has been subjected to long-time
der proper observation to demonstrate its value, and unless
l the proportion in which they are present are known.

neral, more desirable results are obtainable from inert com-
echanically, than from active chemical compounds whose
on change of form through chemical action after addition to

lling substances are more to be relied upon than those whose
repellent action.

ing average quality in sizing of the aggregates and reasonably
in the mixing and placing of the concretes, the addition of
f very finely divided void-filling mineral substances may be
in the production of concrete which, under ordinary conditions
e found impermeable, provided the work-joints are properly
s do not develop on drying, or through change in volume due
nges, or by settlement.

eatment. While external treatment of concrete would not
concrete itself, either naturally or by the addition of water-
was impermeable to water, it has been found in practice that
n, no matter how carefully the concrete itself has been made,
evelop, due to shrinkage in drying out, expansion and contrac-
of temperature and moisture-content, and through settle-
fore, often advisable in important construction to anticipate
e possible occurrence of such cracks by external treatment
ating. Such coating must be sufficiently elastic and cohesive
ks extending through the coating itself. The application of
void-filling liquid washes will not prevent the passage of

Protective Coatings. While protective extraneou
coatings are unnecessary, so far as the major portion
crete is concerned, provided the concrete, either in its
of integral compounds, is made impermeable, they a
where cracks develop in a structure. It is therefore
bination of inert void-filling substances and extraneou
in especially difficult or important work.

Bituminous or Asphaltic Coatings. Considering
asphaltic coatings, the committee finds:

(a) That such protective coatings are often subject
tion with time, and may be attacked by injurious vapo
in solution in the water coming in contact with them

(b) That the most effective method for applying s
setting of a course of impervious brick dipped in bitur
bed of bituminous material, or the application of a su
satisfactory membranous material cemented togethe

(c) That their durability and efficiency are very lar
with which they are applied. Such care refers parti
and preparation of the concrete to insure as dry a surfa
cation of the protective covering, the lapping of all
layers, and their thorough coating with the protect
this method of protection is further desirable because
ings offer resistance to stray electrical currents, the
is referred to in succeeding paragraphs.

Rich Mixtures. So far, the committee has consid
usual proportions, namely, those ranging from 1 of
stone, to 1 of cement, 3 of sand and 6 of stone. It h
permeable concretes could be assured by using mixt
cement. While such practice would probably result
able concrete, it is believed by many that the advan
richer concretes are more subject to check-cracking
volume under changes of conditions of temperature,

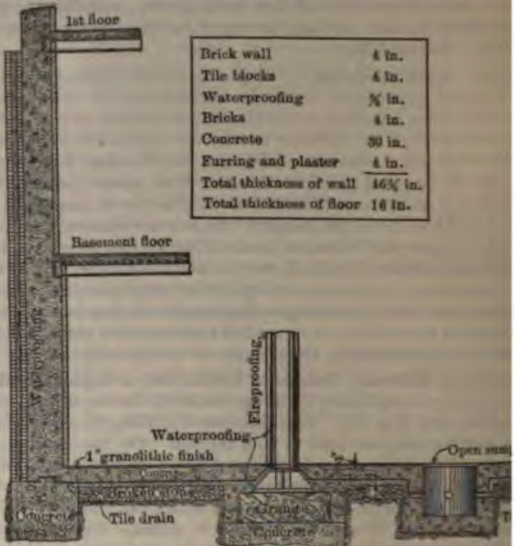
y substitute active cementitious substances for the largely in-
ıg materials previously recommended, thus increasing the strength

Workmanship. In conclusion, the committee would point out
n of waterproofing-compounds or substances can be relied upon
ounteract the effect of bad workmanship, and that the produc-
eable concrete can only be hoped for where there is determined
od workmanship.

s. **Electrical Action.** The production of impermeable concrete
reater importance since the appointment of this committee,
ell-known injurious action of saline or alkaline waters and to the
ble effect of the moisture in concrete occasioning or aggravating
from stray currents. Originally, the question of waterproofing
y the physical troubles resulting from water passing through
t any special consideration of its effect on its durability, other
leaching out of the cement. Recent developments suggest the
owing to the increased conductivity of damp concrete to elec-
such currents, if present, may so affect damp concrete as to seri-
integrity; and this possibility further emphasizes the importance
ndation that no waterproofing-compound of unknown chemical
added to concrete, as recent tests seem to show that the action of
ts is aggravated by the presence of certain solutions.

ng by External Linings of Brick, Tar, or Asphalt, and Felt.
od of waterproofing is the one involving the use of a tar-and-felt
felt seal (Fig. 1). This consists of building first a supporting
porting concrete slab to hold the seal. On the floors, this slab
osed of concrete, 4 in thick. The walls are generally of brick
hick, but occasionally 4-in terra-cotta tiles are used. Upon this
g of tar or asphalt is placed and before this has become cold or
ss of paper, saturated with coal-tar, is laid. This paper receives
coal-tar and asphalt and another layer of paper is placed, the
continued until there are three or more layers of paper with
abbings of the tar or asphalt. For damp-proof work, three layers
our swabbings of tar are usually sufficient. For waterproofing-
han five and usually six layers of paper with from six to seven
r are used. The main walls of the structure are then built against
roofing, and after these are in place, the main concrete basement-
nediately on top of the floor-seal, the idea being to form a con-
proof seal enveloping the entire basement below grade. The
is system consist chiefly in securing perfect laps at all points in
nless extreme care is used and unless there is perfect coöperation
terproofer and the mason-contractor, there is apt to be a break
he seal, usually where the wall-waterproofing is supposed to be
or-work. The disadvantages of this system are due to the fact
not permanent in all soils as the subsurface water frequently
which destroy the seal. Then again, the seal may be easily
he mason-contractor in building his wall against it or in laying
or upon the flat work. The chief disadvantage, however, is that
ng-seal is invariably buried behind a mass of masonry, either
te, which means that should there be a leak, due to either care-
lent, through the waterproofing-seal, it is frequently impossible
not infrequently happens that when a leak has developed in tar-
he actual presence of *the water does not show opposite* the leak,

but following some line of least resistance, appears from 50 to 100 f away from where the actual damage causing the leak occurs. In ac proofing work it is seldom attempted to secure a bottle-tight job w felt. Instead, some system of drainage is installed beneath the seal which is on the floors of the building, and the water is conducted t



Brick wall	4 in.
Tile blocks	4 in.
Waterproofing	⅜ in.
Bricks	4 in.
Concrete	30 in.
Furring and plaster	4 in.
Total thickness of wall	46⅜ in.
Total thickness of floor	16 in.

Fig. 1.* Felt-Waterproofing for Foundations

or other pipes to some central sump from which it is mechanically a sewer. The purpose of the waterproofing in this case, therefore centrate or drive the water to this sump. For shallow cellars an dampproofing-work, this tar-and-felt method is the most economic frequently employed.

Waterproofing by Coating with Water-Proof Cement. Fo difficult work a comparatively new method of waterproofing is (Fig. 2). This consists of placing a coating of water-proof cemen interior surface of the exterior walls of the building and over the u of the concrete floor-slab in the basement or subbasement. Fig. foundation for an engine, the concrete being waterproofed as show is made somewhat larger than the foundation, the extra space b with cinders, dry bricks or terra-cotta blocks, which may be read to allow access to the bed-plate bolts for which hand-holes have the concrete, thus permitting the complete removal of the engine.

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* Reproduced, by permission, from a pamphlet published by The Water pany, New York, and showing the greater thickness of walls and floor req outside-surface brick-and-felt method of waterproofing as compared wit surface waterproof-cement coating. Taken from design for waterproofing i New York building. See, also, Fig. 2.

ınd cushion and a 2-in layer of planks under the engine-foundation. part of the waterproofing but is put in to prevent the communica- ion. Fig. 4 shows reinforced-concrete floors for an engine-room m, the concrete slab being 12 in thick under the former and 24 in

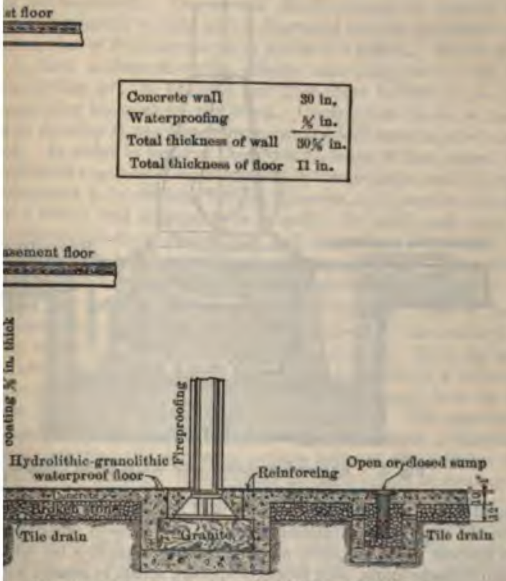

Fig. 2.* Cement Waterproofing for Foundations

e latter. Both floors are covered with a 1-in course of water-proof reinforcement is put in as shown and in sizes and spacing as

12-in slab	24-in slab
ds in two courses	Rods in three courses
4 in on centers, 6 in from	Lowest rods, 3 in on centers, 12 in from surface
5 in on centers, 2 in from	Intermediate rods, 3 in on centers, 7 in from surface
, total area of cross-section in; per square foot of sur-	Upper rods, 6 in on centers, 2 in from surface
	For ten rods, total area of cross-section, 1.4 sq in; per square foot of surface, 4.78 lb

nphlet published by The Waterproofing Company, New York, and show- l thickness of walls and floor required for the inside-surface water-proof of waterproofing. Taken from design for waterproofing of the same in Fig. 1. The walls and floors were put in place in the monolithic form.

There are many compounds advertised to make cement or concret
Besides these, there are water-proof cements manufactured by se
and applied by companies that make a specialty of waterproofi
the many waterproofing-compounds have merit; but the main

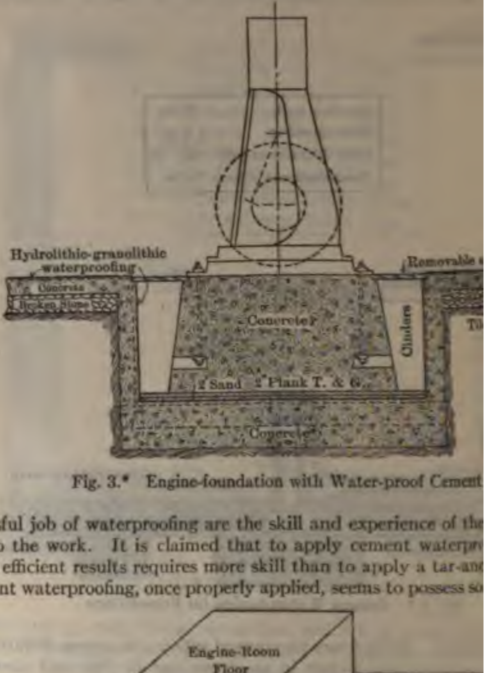

Fig. 3.* Engine-foundation with Water-proof Cement

successful job of waterproofing are the skill and experience of th
who do the work. It is claimed that to apply cement waterp
obtain efficient results requires more skill than to apply a tar-an
a cement waterproofing, once properly applied, seems to possess so

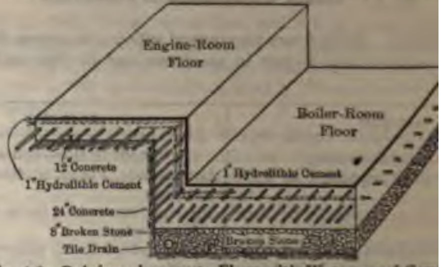

Fig. 4.* Reinforced-concrete Floor with Water-proof Cem

over the older method of tar and felt. One advantage is that th
is accessible, and that if any leaks develop, they are apparent an
and economically repaired by cutting out the old waterproofi
a new coating where the damage exists. Another advantage
cement waterproofing is generally permanent and not damaged

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* Reproduced by permission of The Waterproofing Company, N

solution with water in soil. By the cement method the cost of
rting walls and the concrete supporting slab is eliminated as is
onding cost of the necessary excavation for them; and finally,
ng on the floor serves the double purpose of waterproofing and
, thus saving the cost of the cement finish usually found in
subbasements. One of the disadvantages of cement water-
the material is rigid and is fractured by any settlement of the
raction in the concrete upon which it is placed. Experience has
, that settlement-cracks usually take place before the water-
ctor has left the building and that there is little or no trouble
es after his work is completed. Contraction-cracks in concrete,
o develop at any time within twenty-four months after concrete
. In order to prevent these cracks, users of the cement water-
lopted a system of reinforcement in the concrete, and it is claimed
cement is, in the long run, an economy, as it permits of less con-
a better and stronger floor or wall. On brick and stone walls
xperienced from contraction and expansion. It should be re-
this work is all below grade where contraction and expansion
minimum, regardless of the materials used.

ng by Adding Substances to Cement. This is another
rproofing now being advocated by some. If this method could
e efficient, it would be highly advantageous. It is claimed by
ers of these compounds that in order to secure a water-proof
xample, a certain percentage of the compound is to be mixed
t before it is incorporated in the concrete. The opponents of
im, however, that it is impossible to construct a basement in
ut incurring the danger of serious leaks at the joinings of one
that of another; that leakage at these points of cleavage may
the use of waterproofing-compounds; and that their principal
ey produce a very dense mass of concrete. It is always difficult
crete to new, and if concrete is made water-proof, or, in other
rbent, the difficulty of joining new concrete to a nonabsorbent
crete is increased. This method is effective, however, and is to
d in work which can be carried on without interruption, such, for
all elevator-pits or small swimming-pools, where the concrete
n the morning and completed by night or before any part of the
me to attain its initial set.

FORCE OF THE WIND

tween the Pressure and Velocity of Wind. According to
de in 1890 or thereabouts, by C. F. Marvin, United States
the relation between wind-pressure and velocity is given very
he formula $p = 0.004\ V^2$, where p is the pressure in pounds per
a flat surface normal to the direction of the wind, and V the
wind in miles per hour. Smeaton considered the pressure as
^{2}. The following table, based on Marvin's formula,* is quoted
nd Ketchum.†

---

ormula is written $p = 0.0032\ V^2$ the values in this table will be slightly
hapter XXVII, pages 1052 and 1253, and also page 1308. The
the United States Signal Service is $p = 0.004\ V^2$. The true pressure is
ere between $0.005\ V^2$ and $0.004\ V^2$, near the former for very low veloc-
latter for high velocities.

autwine's Pocket-Book, page *321*.

Table Showing the Force of the Wind

Miles per hour	Feet per minute	Feet per second	Force, in pounds, per square foot	Descri
1	88	1.47	0.004	Hardly perc
2	176	2.93	0.014	Just percept
3	264	4.40	0.036	
4	352	5.87	0.064	Gentle bree
5	440	7.33	0.1	
10	880	14.67	0.4	Pleasant br
15	1 320	22.0	0.9	
20	1 760	29.3	1.6	Brisk gale
25	2 200	26.6	2.5	
30	2 640	44.0	3.6	High wind
35	3 080	51.3	4.9	
40	3 520	58.6	6.4	Very high
45	3 960	66.0	8.1	
50	4 400	73.3	10.0	Storm
60	5 280	88.0	14.4	Great stor
70	6 160	102.7	19.6	
80	7 040	117.3	25.6	Hurricane
100	8 800	146.6	40.0	

COPIES OF TRACINGS

Blue-Prints from Tracings. The following directions* cove ground. The sensitized paper can be procured, all prepared, at artists' materials are sold, so that the process of preparing the pap of chemicals can then be omitted. The materials required are as fol

(1) A board a little larger than the tracing to be copied. T board on which the drawing and tracing are made can always be u

(2) Two or three thicknesses of flannel or other soft white cloth, be smoothly tacked to the board to form a smooth surface, on whic sensitized paper and tracing while printing.

(3) A plate of common double-thick window-glass, of good qua larger than the tracing to be copied. The function of the glass is tracing and sensitized paper closely and smoothly pressed tog printing.

(4) The chemicals for sensitizing the paper. These consist sim parts, by weight, of citrate of iron and ammonia, and red prussia and can be obtained at any drug-store. The price should not 10 cts per ounce for each.

(5) A stone or yellow-glass bottle to keep the solution of the abo in. If there is but little copying to do, an ordinary glass bottle will solution can be freshly made whenever it is wanted for immediate

(6) A shallow earthen dish in which to place the solution when common dinner-plate is as good as anything for this purpose.

(7) A soft paste-brush, about 4 in wide.

(8) Plenty of cold water in which to wash the copies after the exposed to the sunlight. The outlet of an ordinary sink may b placing a piece of paper over it with a weight on top to keep the and the sink filled with water, if the sink is large enough to lay

---

* Taken from The Locomotive.

better to make a water-tight box 5 or 6 in deep, and 6 in wider
the drawing to be copied.
ality of white book-paper.
directions are to be followed:
hemicals in cold water in these proportions: 1 oz of citrate of
ia; 1 oz of red prussiate of potash; and 8 oz of water. They
nto a bottle together and shaken up. Ten minutes will suffice
.

the paper to be sensitized on a smooth table or board, pour a
ion into the earthen dish or plate, and apply a good even coat-
aper with the brush. Then tack the paper to a board by two
and set it in a dark place to dry. One hour is sufficient for
ce the paper, with its sensitized side up, on the board on which
ly tacked the white flannel cloth; lay the tracing to be copied
op of all lay the glass plate, being careful that paper and tracing
and in perfect contact with each other, and lay the whole
sunlight. Between eleven and two o'clock in the summer-time,
rom 6 to 10 minutes will be sufficiently long to expose it; at
longer time will be required. If the location does not admit
, the printing may be done in the shade, or even on a cloudy
to 2½ hours will be required for exposure. A little experience
any one to judge of the proper time for exposure on different
osure, place the print in the sink or trough of water before
wash thoroughly, letting it soak from 3 to 5 minutes. Upon
water, the drawing, hardly visible before, will appear in clear
dark-blue ground. After washing, tack up against the wall,
ent place, by the corners, to dry. This finishes the operation,
ple and thorough. After the copy is dry, it can be written on
pen and a solution of common soda, which makes a white line.

ipe for Making Blue-Prints. The following is an alternative
e given above. The paper should be prepared by floating it
a solution of ferricyanide of potassium (red prussiate of potash),
5 oz. It should then be dried in a dark room, afterwards ex-
he negative until the dark shades have assumed a deep blue
sed in a solution of water, 2 oz, and bichloride of mercury, 1 gr.
be washed, immersed in a hot solution of oxalic acid, 4 drm,
washed again and dried. Where a copy of a drawing is to be
ed paper is placed, sensitive side uppermost, on a flat board
or three thicknesses of blanket or its equivalent. A tracing
made, laid on the sensitized paper and held in place by a sheet
to the board. The sensitized paper is exposed to the sunlight
utes or to a clear sky from 20 to 30 minutes and then removed,
. The only requisite as to paper is that it must stand wash-
aper may be purchased.

opies from Tracings. The directions for making the sensitiz-
in this process are as follows: Dissolve separately, gum arabic,
ater; tartaric acid, 13 dr, in 6 oz 6 dr water; persulphite of iron,
of water. Pour the third solution into the second, stir thor-
pour the resulting mixture into the first, the stirring being
n the mixture is complete add slowly, still stirring, 3 fl oz and
cid perchloride of iron; filter into a bottle and keep in the dark.
l-sized paper, apply a thin, smooth coat of the solution with a
onge, and then *dry in a dark room with moderate heat.* The

Print in a bright light until slightly darker than o
wash for 5 minutes; immerse in a 2½% solution of
obtained; and wash and dry. Blue-prints may
color by immersing in a solution of caustic soda the
5 oz of water, until the blue has changed to oran
washed thoroughly, immersed in a bath of water i
a heaping teaspoonful of tannic acid, rinsed in clea
may be sized for brown-prints by soaking it in a mi
and 5 oz of cold water, rubbed into a cream and mix
5 oz of hot water. The mixture should be boiled 2
to cool before use.

HORSE-POWER,* PULLEYS, GE
AND SHAFTING

Horse-Power. A horse can travel 400 yd at a w
2 min, and at a gallop in 1 min; he occupies at a
average weight is 1 000 lb. An AVERAGE HORSE
25 miles in a day of 8 hr. A DRAUGHT-HORSE can d
weight of carriage included. In a HORSE-MILL a hor
in a second. The diameter of the track should not

A Horse-Power, in Machinery, is estimated at
minute; but as a horse can exert that force but 6
HORSE-POWER is equivalent to that of four horses.

Rules to Determine the Size and Speed of
driving-pulley is called the DRIVER, and the driven
number of teeth in the gears are used instead of the
tions, number of teeth must be substituted whereve

(1) **To Find the Diameter of the Driver,** the dian
revolutions, and also revolutions of driver, being give
of the driven by its revolutions, and divide the pro
the driver; the quotient will give the diameter of the

y its revolutions, and divide the product by the diameter of the
tient will give the revolutions of the driver.

the Revolutions of the Driven, the diameter and revolutions of
the diameter of the driven, being given. Multiply the diameter
y its revolutions, and divide the product by the diameter of the
tient will give the revolutions of the driven.

er Transmitted by Belting. The efficiency of belting to trans-
o turn a wheel or PULLEY, depends upon the width and thickness
arc-contact with the pulley, the position of the belt, whether
ical, or at an angle, and the velocity. The greater the velocity
the belt, the more power it will transmit. A belt running ver-
ed will transmit less power than one running horizontally, but
horse-power capacity of belting only the velocity, width and
lt are usually considered, it being assumed that the pulleys are
and located so that the belt will be nearly horizontal. Belts are
med to be of LEATHER, unless otherwise designated. The term
used to designate a belt made of a single thickness of cowhide
UBLE BELT is made by cementing and riveting together two
eather. There is no standard thickness for either single or double

y rules have been given for determining the horse-power that
smit.* Those commonly used are:

le Belts. Multiply the width, in inches, by the velocity in feet
divide by 1 000.

le Belts. Multiply the width by the velocity and divide by 700.
he number of horse-powers.

ties give divisors of 800 and 733 for single belts, and 550 and 513
. For the velocity of the belt multiply the number of revolutions
ither pulley by the circumference of that pulley.

lting. For continuous use a double belt is the most economical
, except on very small pulleys or for very light duty. Triplex
belts are sometimes used for very heavy duty, but such belts are
carried in stock. Single belts should always be used with the
the pulley. The belt-speed for maximum economy should be
4 500 ft per minute. IDLER-PULLEYS work most satisfactorily
the slack side of the belt about one-quarter way from the driv-
lts are more durable and work more satisfactorily when made
ck than when made wide and thin. As belts increase in width
o be made thicker. For dynamo-work or electric motors the
t should be fastened together by splicing and cementing instead
For all other cases the ends are fastened by hooks or lacing.
cleaned and greased every 5 to 6 months.

om Center to Center of Shafts.* In locating shafts that are
with each other by belts, care should be taken to separate them
tance. This distance should be such as to allow a gentle sag to
n motion.

eral rule may be stated thus: Where narrow belts are to be run
ys, 15 ft is a good average, the belt having a sag of from 1½ to
imum distance between shafts is about 10 ft. For larger belts,
er pulleys, a distance of from 20 to 25 ft does well, with a sag

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ion of belting, belt-dressing, care of belting, shafting, etc., see Kent's
eers' *Pocket-Book.*

of from 2½ to 4 in. For main belts, working on very large pulleys, th
should be from 25 to 30 ft, the belts working well with a sag of from
If too great a distance is attempted, the belt will have an unsteady
motion, which will destroy both the belt and the machinery.

Arrangement of Belts and Pulleys.* If possible to avoid it,
shafts should never be placed one directly over the other, as in such
belt must be kept very tight to do the work. For this purpose belts
carefully selected of well-stretched leather. It is desirable that the
the belt with the floor should not exceed 45°. It is also desirable to
shafting and machinery so that belts will run off from each shaft in
directions, as this arrangement will relieve the bearings from the fri
would result if all pulled one way on the shaft. If possible, machiner
be so placed that the direction of the belt-motion will be from the to
driving to the top of the driven pulley, so that the sag will increase th
contact. The pulley should be a little wider than the belt required
work, and should have a crowning face, except where the belt is to be
The motion of driving should run with and not against the la
belts.

Rubber Belts are cheaper than leather belts and should always b
wet places, but for ordinary use in dry places they are not as durable
belts. They should always be kept free from grease or animal oils. If
their inside surfaces should be moistened with boiled linseed-oil.
chalk, sprinkled on over the oil, will help the belt.

Rule for Finding the Lengths of Belts. Add the diameter o
pulleys together, multiply by 3¼, divide the product by 2, add to th
twice the distance between the center of the shafts, and the sum
required length.

The Horse-Power that Shafting will Transmit

Diameter of shaft		Revolutions per minute					
		100	150	200	250	300	35
in	16th	H.P.	H.P.	H.P.	H.P.	H.P.	H.P.
0	15	1.2	1.7	2.4	3.1	3.6	4.3
1	3	2.4	3.7	4.9	6.1	7.3	8.5
1	7	4.3	6.4	8.5	10.5	12.7	14.8
1	11	6.7	10.1	13.4	16.7	20.1	23.4
1	15	10.0	15.0	20.0	25.0	30.0	35.0
2	3	14.3	21.4	28.5	35.6	42.7	49.8
2	7	19.5	29.3	39.0	48.7	58.5	68.2
2	11	26.0	39.0	52.0	65.0	78.0	87.0
2	15	33.8	50.6	67.5	84.4	101.3	118.2
3	3	43.0	64.4	85.8	107.3	128.7	150.3
3	7	53.6	79.4	107.2	134.0	158.8	187.6
3	11	65.9	97.9	121.8	164.8	195.7	230.7
3	15	80.0	120.0	160.0	200.0	240.0	280.0
4	7	113.9	170.8	227.8	284.7	341.7	398.6
4	15	156.3	234.4	312.5	390.6	468.7	546.8

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* See Kent's Mechanical Engineers' Pocket-Book.

CHAIN-BLOCKS, HOISTS, HOOKS, ETC.

neral Description. These are portable hoisting-devices which enable one o raise a very heavy load and which sustain the load at any point. In l, they resemble pulleys operated by chains. Since the invention of the ntial pulley-block by T. A. Weston, about the year 1863, chain-blocks ome into very general use for economical hoisting, particularly where it is d to hold the load at any point. Chain-blocks are of three general classes:
The Differential Block. This is the original and the simplest and cheap- m of self-sustaining pulley;
The Screw-Block or Worm-Geared Block. Of these, the Yale & Towne block is the most efficient type;
The Triplex Block. This is spur-geared.
erential and worm-geared blocks of all kinds depend upon friction to pre- the load from running down. In the triplex block a separate device is uced which automatically holds the load safely, and yet enables it to be ed with slight effort and at high velocity but without acceleration or danger. is the most efficient of all chain-blocks, and the most economical wherever work is wanted and economy in time and labor sought. For information the kind of block best adapted to any particular service, the manufacturers d be consulted. The following data on the power and efficiency of chain- s were supplied by the Yale & Towne Manufacturing Company.

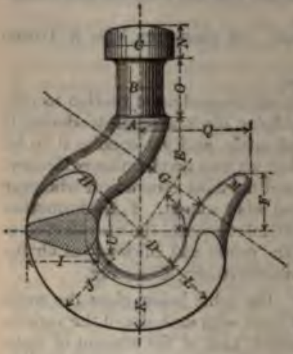
wer and Efficiency of Chain-Hoists. The table below gives the work done by the operator at the hand-pulling chain with each size of the various s of chain-blocks in lifting the stated capacity, that is, the amount of work ulling required to lift this load ONE FOOT by stating the force exerted in ds and the distance in feet of operating-chains to be pulled. The product hese two factors determines the efficiency of the block and the ease and speed oisting.

Work Done by Operator with Chain-Blocks

Capacity, tons	Triplex spur-geared, lb ft	Duplex worm-geared, lb ft	Differential lb ft
½	62× 21	68× 40	122×24
1	82× 31	87× 59	216×30
1½	110× 35	94× 80	246×36
2	120× 42	115× 93	308×42
3	114× 69	132×126	557×38
4	124× 84	142×155	....
5	110×126	145×195	....
6	130×126	145×252	....
8	135×168	160×310	....
10	140×210	160×390	....
12	130×126	....	....
16	135×168	....	....
20	140×210	....	....

hese blocks have two hand-chains. The figures give the number of feet operated on each hand-chain. A man cannot pull more than his own ht on the operating chains, and can pull faster in proportion as the pull ired is lighter. The maximum pull usually required of one man is 82 lb, he will do more *work with less fatigue if the hand-chain pull* is not over

40 lb, because he can then pull the chain hand over hand a little as fast as he could when pulling twice as hard. When the h less than 20 lb the speed of hoisting an equal load is diminished, is tired by moving his arms too rapidly, and cannot do as muc heavier pull. The best result is obtained by using a chain- a capacity of double the usual load. The operator then work vantage with average loads, and occasional heavy loads are easi out overstraining either the operator or the chain-block, which used beyond its capacity fo ing the chain so that it smoothly.



Proportions of Hooks. of manufacture hooks of d made from some regular c of round iron. The basis, in each case is, therefore, iron of which the hook is t it is indicated by the dime diagram. The dimension assumed. The other dimen by the formulas, are thos preserving a proper bearin interior of the hook for the which may be passed throu greatest resistance to spr ultimate rupture which t material in the original bar admits of. The symbol Δ is used i to indicate the NOMINAL CAPACITY of the hook in tons of 2 000 lb. which determine the lines of the other parts of the hooks of the s as follows, all the measurements being expressed in inches:

$D = 0.5\,\Delta + 1.25$
$E = 0.64\,\Delta + 1.60$
$F = 0.33\,\Delta + 0.85$
$H = 1.08\,A$
$I = 1.33\,A$
$J = 1.20\,A$
$K = 1.13\,A$

$G = 0.75\,D$
$O = 0.363\,\Delta +$
$Q = 0.64\,\Delta +$
$L = 1.05\,A$
$M = 0.50\,A$
$N = 0.85\,B -$
$U = 0.866\,A$

Example. To find the dimension D, for a 2-ton hook. The fo

$$D = 0.5\,\Delta + 1.25$$

and as $\Delta = 2$, the dimension D by the formula is found to be 2¼ i sions A, are necessarily based upon the ordinary merchant sizes The sizes which it has been found best to select are the following:

Capacities of hooks	⅛	¼	½	1	1½	3	2	4	5	6	8
Dimension A	⅝	11⁄16	¾	1 1⁄16	1¼	1⅜	1¾	2	2¼	2½	2¾

The formulas which give the sections of the hook at the sev all expressed in terms of A, and can therefore be readily ascert ence to the foregoing scale.

---

* By Henry R. Towne, in his Treatise on Cranes, which includes the tensive experimental and mathematical investigation.

Example. To find the dimension I, in a 2-ton hook. The formula is

$$I = 1.33\,A$$

d for a 2-ton hook, $A = 1\frac{3}{8}$ in. Therefore I, in a 2-ton hook, is found to be is in.

Manner of Failure of Hooks. Experiment has shown that hooks made cording to the above formulas will give way first by the opening of the jaw, ich, however, will not occur except with a load much in excess of the nominal pacity of the hook. This yielding of the hook when overloaded becomes a urce of safety, as it constitutes a signal of danger which cannot easily be overoked, and which must proceed to a considerable length before rupture occurs d the load is dropped. A comparison of these hooks with most of those in dinary use shows that the latter are, as a rule, badly proportioned, and frequently dangerously weak.

BELLS

Dimensions and Weights of Church-Bells

Manufactured by Meneely Bell Company, Troy, N. Y.

Bells		Mountings			
Weights, lb	Medium tones	Diameters, in	Sizes of frames, outside, in	Diameters of wheels, ft	in
400	D	27	42×42	4	4
450	C♯	28	42×42	4	4
500	C	29	45×47	4	4
550	C	30	45×47	4	4
600	B	31	45×47	4	9
700	B	33	48×48	5	6
800	B♭	34	48×54	5	6
900	A	36	54×54	5	9
1 000	A	37	54×54	5	9
1 100	A	38	54×59	5	9
1 200	A♭	39	56×59	6	3
1 300	A♭	40	56×59	6	3
1 400	G	41	60×60	6	6
1 500	G	42	60×60	6	6
1 600	G	43	60×60	6	6
1 800	F♯	45	65×68	7	
2 000	F	46	65×68	7	
2 100	F	47	65×68	7	
2 300	E	49	70×72	7	6
2 500	E	50	70×72	7	6
2 800	E♭	51	74×78	8	
3 000	E♭	53	74×78	8	
3 500	D	56	74×78	8	6
4 000	C♯	58	78×81	9	
4 500	C	61	78×81	9	
5 000	C	63	84×84	9	
5 500	B	65	84×84	9	
6 000	B♭	67	84×84	9	6
6 500	B♭	68	90×90	9	6
7 000*	B♭	69	101×90	9	6

A notable example of a 7 000-lb bell is the large bell of the peal in the tower of the tropolitan Life Insurance Building, in New York.

he actual weights *usually exceed the patterns, noted above, from 2 to 3%.*

Meneely School-Bells

Bells		Mountings
Weights, lb	Diameters, in	Sizes of frames, outside, ft in ft in
100	17	2 6 × 2 8
125	18½	2 6 × 2 8
150	19½	2 6 × 2 8
200	21½	2 8 × 3 0
250	23	3 0 × 3 2
300	24½	3 0 × 3 4
350	26	3 0 × 3 4

Sizes of Rope for Bells

	Diameter, i
For bells of less than 500 lb.	½
For bells of 500 to 800 lb	⅝
For bells of 800 to 1 800 lb.	¾
For bells above 1 800 lb.	¾ to 1

The Largest Bells in the World *

Names and locations of bells	Date cast	Actual vibration	Key-note	Diameter, in	Sound-bow	
					Inches	Strok
Moscow, Tzar Kolokol †.	1733	74	D	272	23	0.84
Burmah, Mingoon	....	94	F♯	203?	16?	0.80
Moscow, St. Ivan's	1819	105	G♯	185	14.75	0.80
Pekin, Great Bell	....	......	....	156	......	......
Burmah, Maha Ganda	....	125	B	155	12.5	0.8
Nishni Novgorod	....	125	B	151	12	0.8
Moscow, Church of Redeemer	1879	141	C♯	136.3?	10.6	0.8
Nankin, China	....	......	....	112	......	.....
London, St. Paul's.	1881	157	E♭	114.25	8.75	0.7
Olmutz, Bohemia	....	157	E♭	121	9.125	0.7
Vienna, Austria	1711	157	E♭	118	9.5	0.8
Westminster, London	1856	166	E	113.5	9.375	0.8
Erfurt, Saxony	1487	176	F	103.6	9.75	0.
Notre Dame, Paris	1680	166	E	103	7.5	0.
Montreal, Canada	1847	176	F	103	7.8	0.
York, England	1845	187	F♯	100	8	0.
St. Peter's, Rome	1786	187	F♯	97.25	7.5	0
Great Tom, Oxford	1680	210	G♯	84	6.125	0.
Cologne, Germany	1477	198	G	95	7.2	0.
Brussels, Belgium	....	210	G♯	95.81	7.75	0.
State-house, Philadelphia	1875	198	G	88	6.375	0.
Lincoln, England	1834	210	G♯	82.85	6	0
St. Paul's, London	1716	222	A	81	6.08	0.
Exeter, England	1675	210	G♯	76	5	0.
Old Lincoln, England	1610	249	B	75.5	5.94	0
Westminster, London	1857	249	B	72	5.75	0.

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* John W. Nystrom, in the Journal of the Franklin Institute, Philad

† This bell is fractured and has not been rung for many years.

MBOLS FOR THE APOSTLES AND SAINTS

e constant occurrence of symbols in the edifices of the Middle Ages of the cathedrals of the present day, the following list of symbols, as attached to the apostles and saints, may be found useful:

Holy Apostles

Bears a key, or two keys with different wards.
. Leans on a cross so called from him; called by heralds the saltire.
e Evangelist. With a chalice, in which is a winged serpent. When mbol is used, the eagle, another symbol of him, is never given.
omew. With a flaying-knife.
he Less. A fuller's staff bearing a small square banner.
he Greater. A pilgrim's staff, hat and escalop-shell.
s. An arrow, or with a long staff.
A long saw.
A club.
as. A hatchet.
Leans on a spear or has a long cross in the shape of a T.
w. A knife or dagger.
A winged lion.
A bull.
An eagle.
An elevated sword, or two swords in saltire.
e Baptist. An Agnus Dei.
. With stones in his lap.

Saints

A lamb at her feet.
With an organ.
t. With an anchor.
Preaching on a hill.
With his head in his hands.
. With the dragon.
s. With three naked children in a tub, in the end whereof rests his al staff.
. On the rack.

LAR OF ADVICE ON PROFESSIONAL PRACTICE, HE AMERICAN INSTITUTE OF ARCHITECTS*

ctory. The American Institute of Architects, seeking to maintain a rd of practice and conduct on the part of its members as a safeguard ortant financial, technical and esthetic interests entrusted to them, following advice relative to professional practice: The profession of e calls for men of the highest integrity, business capacity and artistic he architect is entrusted with financial undertakings in which his purpose must be above suspicion; he acts as professional adviser to nd his advice must be absolutely disinterested; he is charged with e of judicial functions as between client and contractors and must

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erican Institute of Architects, Document 107, Washington, D. C., May 10, rinted by permission. This circular relates to the principles of professional the canons of ethics.

between his client and a contractor by the terms of which
the official interpreter of its conditions and the judge
additional relation is created under which it is incumbe
to side neither with client nor contractor, but to use his
tract to enforce its faithful performance by both partie
architect's payment comes from the client does not inval
act with impartiality to both parties.

(2) **On Preliminary Drawings and Estimates.** Th
set should impress upon the client the importance of
preparation of drawings and specifications. It is the du
make or secure PRELIMINARY ESTIMATES when requested,
the client with their conditional character and inform h
final figures can be had only from complete and final draw
If an unconditional limit of cost be imposed before such
estimated, the architect must be free to make such adjus
necessary. Since the architect should assume no respor
vent him from giving his client disinterested advice, he s
otherwise, guarantee any estimate or contract.

(3) **On Superintendence and Expert Services.**
simplest, it is to the interest of the owner to employ a SUP
OF THE WORKS. In many engineering problems and in cert
problems, it is to his interest to have the services of s
architect should so inform him. The experience and sp
architect make it to the advantage of the owner that th
paid by the owner, should be selected by the architect und
are to work.

(4) **On the Architect's Charges.** The SCHEDULE
American Institute of Architects is recognized as a proper
The locality or the nature of the work, the quality of se
the skill of the practitioner or other circumstances frequ
charge than that indicated by the schedule.

(5) **On Payment for Expert Service.** The architec

g that none but trustworthy bidders be invited and that the award be
nly to contractors who are reliable and competent, the architect protects
erests of his client.

On Duties to the Contractor. As the architect decides whether or
intent of his plans and specifications is properly carried out, he should
ecial care to see that these drawings and specifications are complete and
te, and he should never call upon the contractor to make good oversights
rs in them nor attempt to shirk responsibility by indefinite clauses in the
ct or specifications.

On Engaging in the Building Trades. The architect should not
y or indirectly engage in any of the BUILDING TRADES. If he has any
al interest in any building material or device, he should not specify or
without the knowledge and approval of his client.

On Accepting Commissions or Favors. The architect should not
any COMMISSION or any substantial service from a contractor or from any
ted person other than his client.

On Encouraging Good Workmanship. The large powers with which
hitect is invested should be used with judgment. While he must condemn
rk, he should commend good work. Intelligent initiative on the part of
en and workmen should be recognized and encouraged and the architect
make evident his appreciation of the dignity of the ARTISAN'S FUNCTION.

On Offering Services Gratuitously. The seeking out of a possible
and the offering to him of professional services on approval and WITHOUT
NSATION, unless warranted by personal or previous business relations,
o lower the dignity and standing of the profession and is to be condemned.

On Advertising. ADVERTISING tends to lower the dignity of the pro-
and is therefore condemned.

On Signing Buildings and Use of Titles. The display of the archi-
name upon a building under construction is condemned, but the unob-
SIGNATURE OF BUILDINGS after completion has the approval of the
te. The use of INITIALS designating membership in the Institute is proper
ection with any professional service and is to be encouraged as helping
e known the nature of the honor they imply.

On Competitions. An architect should not take part in a competition
MPETITOR or JUROR unless the competition is to be conducted according
best practice and usage of the profession, as evidenced by its having
d the approval of the Institute, nor should he continue to act as PRO-
NAL ADVISER after it has been determined that the program cannot be so
as to receive such approval. When an architect has been authorized to
sketches for a given project, no other architect should submit sketches
until the owner has taken definite action on the first sketches, since, as
the second architect is concerned, a competition is thus established.
as an authorized competitor, an architect may not attempt to secure
or which a competition has been instituted. He may not attempt to
ce the award in a competition in which he has submitted drawings. He
t accept the commission to do the work for which a competition has been
ted if he has acted in an advisory capacity either in drawing the program
aking the award.

On Injuring Others. An architect should not falsely or maliciously
directly or indirectly, the professional reputation, prospects or business
llow architect.

ation is essential for the practice of architecture, archite
insist that it should rest upon a broad foundation of gene

(18) **On Duties to the Public and to Building A**
tect should be mindful of the public welfare and shou
movements for public betterment in which his special t
qualify him to act. He should not, even under his clien
in or encourage any practices contrary to law or hostile
for as he is not obliged to accept a given piece of work, he
he has but followed his client's instructions, escape the c
to his acts. An architect should support all public offi
of building in the rightful performance of their legal du
fully comply with all building laws and regulations, and
him unwise or unfair, he should endeavor to have them a

(19) **On Professional Qualifications.** The public b
that he who bears the TITLE OF ARCHITECT has the knowle
for the proper invention, illustration and supervision of
which he may undertake. Such qualifications alone jus
the title of architect.

The Canons of Ethics *

The Following Canons are Adopted by The A
Architects as a general guide, yet the enumeration of p
not be construed as a denial of the existence of others equal
not specially mentioned. It should also be noted tha
indicate offenses of greatly varying degrees of gravity. I
an architect

(1) To engage directly or indirectly in any of the buil
(2) To guarantee an estimate or contract by bond or o
(3) To accept any commission or substantial service
from any interested party other than the owner.
(4) To advertise.

ttempt to influence, either directly or indirectly, the award of a com-
which he is a competitor.
ccept the commission to do the work for which a competition has been
if he has acted in an advisory capacity, either in drawing the pro-
r in making the award.
injure falsely or maliciously, directly or indirectly, the professional
, prospects, or business of a fellow architect.
undertake a commission while the claim for compensation, or damages,
f an architect previously employed and whose employment has been
d remains unsatisfied, until such claim has been referred to arbitration
as been joined at law, or unless the architect previously employed
press his claim legally.
attempt to supplant a fellow architect after definite steps have been
ard his employment, that is, by submitting sketches for a project for
ther architect has been authorized to submit sketches.
compete knowingly with a fellow architect for employment on the
rofessional charges.

ional Practice of Architects. Schedule of Proper Minimum Charges *

architect's professional services consist of the necessary conferences,
ration of preliminary studies, working drawings, specifications, large
full-size detail drawings, and of the general direction and supervision
rk, for which, except as hereinafter mentioned, the minimum charge,
n the total cost † of the work complete, is 6%.
residential work, alterations to existing buildings, monuments, furni-
rative and cabinetwork and landscape-architecture, it is proper to
igher charge than above indicated.
architect is entitled to compensation for articles purchased under his
even though not designed by him.
n operation is conducted under separate contracts, rather than under
contract, it is proper to charge a special fee in addition to the charges
d elsewhere in this schedule.
ere the architect is not otherwise retained, consultation-fees for pro-
dvice are to be paid in proportion to the importance of the question
nd services rendered.
ere heating, ventilating, mechanical, structural, electrical and sanitary
are of such a nature as to require the services of a specialist, the owner
for such services. Chemical and mechanical tests and surveys, when
are to be paid for by the owner.
cessary traveling expenses are to be paid by the owner.
after a definite scheme has been approved, changes in drawings, speci-
or other documents are required by the owner; or if the architect
xtra labor or expense by the delinquency or insolvency of a contractor,
ect shall be paid for such additional services and expense.
yments to the Architect are due as his work progresses in the following
pon completion of the preliminary studies, one-fifth of the entire fee;

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vised at the Washington, D. C., Convention, December 15–17, 1908. Re-
permission.

tal cost is to be interpreted as the cost of all materials and labor necessary to
he work, plus contractors' profits and expenses, as such cost would be if all
ere new and all labor fully paid, at market prices current when the work was

in process of erection, completion or alteration, as he find
whether it is being executed in general conformity with h
cations or directions. He has authority to reject any p
does not so conform and to order its removal and re
authority to act in emergencies that may arise in the co
order necessary changes, and to define the intent and m
and specifications. On operations where a clerk of the w
of construction is required, the architect shall employ
owner's expense.

(12) Drawings and specifications, as instruments of se
of the architect.

ARCHITECTURAL COMPETIT

This Circular of Advice furnishes information as t
conducting architectural competitions and states the co
requisite to participation in them by members of The
Architects. It does not apply to competitions for work
than in the United States, its territories and possessions.

The Attitude of The American Institute of Archit
Since its foundation, more than fifty years ago (1857), T
of Architects has given much attention to the conduct o
PETITIONS. These contests, instituted when the direct s
could not be made, were for many years conducted wit
and often in disregard of the interests both of the owner
The owner, totally unfamiliar with the intricacies of the

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* The editor is advised (February, 1915) by the chairman of th
of Charges of The American Institute of Architects, that in ca
the work, or in case the architect should be discharged for a
superintend the work, this charge for full-size details is to be
named as compensation for working drawings and specifications

† The American Institute of Architects, Document 114.

assistance, to prepare the programme, laying down, or more frequently
ules to govern procedure. With the growth of the country, the in-
xpenditures for public and private buildings, and the increase in the
architects, all the evils of ill-regulated competitions became more
Programmes varied from loose and careless forms, difficult to under-
often open to the suspicion that only the initiated knew what they
over-elaborate ones necessitating useless study of details and needless
Those instituting the competition often had no legal authority to pay
titors, still less to employ the winner. There was great economic
total cost of participation exceeding the total net profit accruing to the
from work secured through competitions. Architects have learned
utcome of a competition, unless governed by well-defined agreements,
matter of chance. The owner has, to be sure, a choice of designs, but
ore likely to make the wisest selection or to obtain the best building
selects his architect directly, guided by the results previously achieved
n he is considering. When a competition is necessary or desirable
be of such form as to establish equitable relations between the owner
mpetitors. To insure this:

REQUIREMENTS should be clear and definite, and the statement of
e it must be in technical terms, should be drawn by one familiar with
s.

COMPETENCY of all competing should be assured. The drawings sub-
competition are evidence, only in part, of the ability of the architect
the building. The owner, for his own protection, should admit to
tition only those to whom he would be willing to entrust the work;
men of known honesty and competence.

AGREEMENT between the owner and the competitors should be defi-
comes a plain statement of business relations.

JUDGMENT should be based on knowledge, and since ideas presented
of drawings are intelligible only to a trained mind, judgment should
dered until the owner has received competent technical advice as to
of those ideas.

up: To insure the best results, a competition should have (1) a clear
e, (2) competent competitors, (3) a business agreement, (4) a fair

years ago (1900) many competitions had none of these provisions and
ll of them. The commonest form of competition was one that was
l, had a programme prepared by a layman, was judged by the owner
ofessional assistance, contained no agreement, and made no provision
te the incompetent. All this demanded correction. The Institute,
neans of reform, perceived at once that its relation to the owner could
a ADVISORY one. It might advise him how to hold a competition,
uld go no further. To architects in general the Institute could
resume to offer even its advice, but being a professional body
ith maintaining ethical standards among its own members, its duty
that they did not take part in competitions that fell below a reason-
ard.

therefore, voted in convention of the Institute that members should
take part in competitions only when their terms had received the
OF THE INSTITUTE. Thereupon the Institute fully stated the prin-
h should govern competitions and defined the conditions prerequisite
ng of its approval. These are contained in the CIRCULAR OF ADVICE
ving, which is intended as a guide to all who are interested in com-
Committees *of the Institute throughout the country* are authorized

He who raises his programme to that standard has no d
services of architects of the greatest ability. Even in
Institute first made its firm stand against the abuses of
of that action has been far greater than could have bee
altogether eliminated ill-regulated competitions, but it h
number, and it is safe to say that no competition of pr
conducted except in accordance with the principles state
CULAR OF ADVICE:

A Circular of Advice and Information Relative Architectural Competitions

Competitions are instituted to enable the OWNER * t
through comparison of the designs submitted. The Amer
tects, believing that the interests of both owner and com
by fair and equitable agreements between them, issues t
MENT OF THE PRINCIPLES which should underlie such agre
does not assume to dictate the owner's course in conduc
aims to assist him by advising the adoption of such met
proved to be just and wise. So important, however, do
methods appear to architects that members of the Institu
competitions except under conditions based on this circu
forth in Articles (16) and (18).

(1) **On Competitions in General.** A competition ex
architects prepare sketches at the same time for the same
tect who prepares drawings for comparison in problems
cational nature, where the problem does not involve a defi
operation, shall be held as having taken part in a competi
ing of this circular of advice.

(2) **On the Employment of a Professional Advi**
shall be instituted without the aid of a competent ADVIS
architect of the highest standing and his selection shoul
step. He must be chosen with the greatest care, as the suc

'e to advise those who hold the competition as to its form and
p the programme, to advise in choosing the competitors, to
tions, and to conduct the competition.

ɔrms of Competition. The following forms of competition

his form, participation is limited to a certain number of archi-
s should be stated in the programme and to any one of whom
ng to entrust the work. In a LIMITED COMPETITION the com-
:hosen (a) from among architects whose ability is so evident
ıquiry into their qualifications is needed, or (b) from among
ake application accompanied by evidence of their education
The limited form has the advantage that the owner and the
ıer may meet competitors and discuss the terms of the com-
m before the issuance of the programme. Form (a) is the
t direct form of competition.
stitute believes that a competition OPEN TO ALL who wish to
ıt regard to their qualifications is detrimental to the interests
l of architects. It will, therefore, give its approval to that form
:ted in two stages, since by that means alone it is possible to
of submission while safeguarding the owner's interests against
inner of a person lacking the qualifications set forth in Article
ıis form there is a FIRST STAGE open to all, in which the com-
are of the slightest nature, involving only the fundamental
on. These drawings are accompanied by evidence of the com-
n and experience. From the first stage a small number who
strated their competence to design the work and to carry it
xecution are chosen to take part in a FINAL and strictly ANONY-
ving competitive drawings of the type indicated in Article (8)

ɂualification of Competitors. The interests of the owner
prejudiced by admitting to a limited competition or to the
n open competition any architect who has not established to
f the owner his competence to design and execute the work.
ged that by admitting all who wish to take part some unknown
ıer may be found. If the object of a competition were a set
easoning might be valid. But sketches give no evidence that
the matured artistic ability to fulfil their promise, or that he
knowledge necessary to control the design of the highly com-
d equipment of a modern building, or that he has executive
fairs, or the force to compel the proper execution of contracts.
ten been made to defend the owner's interests by associating
bility with one lacking in experience. These have generally
:. As the owner should feel bound, not only legally, but in
retain as his architect the competitor to whom the award is
ial that the competitors in a limited competition, or in the
open competition, should be selected with the greatest care in
the professional adviser, and that there should be included
architects in whose ability and integrity the owner has abso-
ıd to any one of whom he is willing to entrust the work.

umber of Competitors. Experience has demonstrated that
ıANY COMPETITORS is detrimental to the success of a competi-
e are many, each knows that he has but a slight chance of
therefore less aroused to his best effort than when there are but

limit of cost. The programme should neither require nor pern to furnish their own or builders' estimates of the cost of executi accordance with their designs. Such estimates are singularly the cubage be properly limited they are unnecessary.

(8) **On the Jury of Award.** To insure a wise and just aw tect the interests of both the owner and the competitors, the co ings should be submitted to a JURY so chosen as to secure expert freedom from personal bias. Such a jury thoroughly unders explain the intent of the drawings. It discovers from them skill in design, arrangement and construction. Because of its tr its advice as to the merits of the designs submitted is of the high owner. The jury must consist of at least three members, one and a majority of whom should, be PRACTICING ARCHITECTS. members of the jury may be chosen by the competitors. It is th JURY to study carefully the programme and all conditions relating and the competition before examining the designs submitted; to or recommend an award in favor of the author of any design that the conditions distinctly stated as mandatory in the programme; time to the careful study of the designs; and to render a deci mature consideration. The jury should see to it that a copy of its every competitor. The professional adviser should not be a jury, as his judgment is apt to be influenced by his previous study

(9) **On the Competitive Drawings.** The purpose of an arc petition is not to secure fully developed plans, but such evidence ing the essential elements of the problem as will assist in the se ARCHITECT. The drawings should, therefore, be as few in numbe in character as will express the general design of the building. A does not need elaborate drawings.

(10) **On the Programme.** The programme should contain conduct of the competition, instructions for competitors and the agreement between the owner and the competitors. Uniform co competitors are fundamental to the proper conduct of competiti

should be as few as possible, and should relate only to matters which t be left to the discretion of the competitors. It is difficult to summarize the programme, but it should at least:

Name the owner of the structure forming the subject of the competition, ate whether the owner institutes the competition personally or through entatives; if the latter, name the representatives, state how their author- derived, and define its scope.

State the kind of competition to be instituted, and in limited competitions the competitors; or in open competitions, if the competition is limited aphically or otherwise, state the limits.

Fix a time and place for the receipt of the designs. The time should not ered except with the unanimous consent of the competitors.

Furnish exact information as to the site.

State the desired accommodation, avoiding detail.

State the cost if it be fixed or, better, limit the cubic contents.

Fix uniform requirements for the drawings, giving the number, the scale les, and the method of rendering.

Forbid the submission of more than one design by any one competitor.

Provide a method for insuring anonymity of submission.

Name the members of the jury or provide for their selection. Define powers and duties. If for legal reasons the jury may not make the final l, state such reasons and in whom such power is vested.

Provide that no award shall be made in favor of any design until the jury have certified that it does not violate any mandatory requirement of the amme.

Provide that during the competition there shall be no communication ve to it between any competitor and the owner, his representatives, or any ber of the jury, and that any communication with the professional adviser be in writing. Provide also that any information, whether in answer to communications or not, shall be given in writing simultaneously to all etitors. Set a date after which no questions will be answered.

State the number and amount of payments to competitors.

Provide that the professional adviser shall send a report of the competi- o each competitor, including therein the report of the jury.

Provide that no drawing shall be exhibited or made public until after the l of the jury.

Provide for the return of unsuccessful drawings to their respective authors a reasonable time.

Provide that nothing original in any of the unsuccessful designs shall be without consent of, and compensation to, the author of the design in which pears.

Include the contract between the owner and the competitors.

Include the contract between the owner and the architect receiving the d.

) **On the Agreement.** An owner who institutes a competition assumes ral obligation to retain one of the competitors as his architect. In order architects invited to compete may determine whether they will take part essential that they should know the terms upon which the winner will be oyed; and it is of the utmost importance to the owner that those terms ld be so clearly defined that no disagreement as to their meaning can arise the award is made. Unless they be so defined, delay is likely to occur and reements to arise at a time when a complete understanding between owner architect is most important for the welfare of the work. Therefore, there

principal by contracts with the competitors, they should seek a
authority before issuing an invitation. If authority cannot leg
to the jury to make the award, that fact should be stated, and t
in which such authority is vested.

(14) **On the Conduct of the Owner.** In order to mai
impartiality toward all competitors, the owner, his representati
nected with the enterprise should, as soon as a professional a
appointed, refrain from holding any communication in regard
with any architect except the adviser or the jurors. The mee
petitors described in Article (3) is of course an exception.

(15) **On the Conduct of Architects.** An architect shou
in any way, except as a duly authorized competitor, to secure w
competition is in progress, nor should he attempt to influence,
or indirectly, the award in a competition in which he is a compet
tect should not accept the commission to do the work for whic
has been instituted if he has acted in an advisory capacity, either
programme or making the award. An architect should not sub
tion a design which has not been produced in his own office or
direction. No competitor should enter into association with an
except with the consent of the owner. If such associates shoul
petition, their association should continue until the completio
thus won. During the competition, no competitor should hold
cation relative to it with the owner, his representatives or any
jury, nor should he hold any communication with the professional
it be in writing. When an architect has been authorized to s
for a given project, no other architect should submit sketches
owner has taken definite action on the first sketches, since, as fa
architect is concerned, a competition is thus established.

(16) **On the Participation of Members of the Institute**
The American Institute of Architects do not take part as compe
in any competition the programme of which has not received the
of the Institute, nor does a member continue to act as profession
it has been determined that the programme cannot be

MMITTEE for the territory of each chapter, representing the
relation to competitions for work to be erected within such

of the chapter is EX-OFFICIO chairman of the subcommittee,
bers of which he appoints. The subcommittees derive their
the Institute and not from the chapters. An appeal from the
ubcommittee may be made to the standing committee. The
ttee may approve, modify or annul the decision of a subcom-

stitute's Approval of the Programme. The approval of the
given to a programme unless it meets the following essential

e be a professional adviser.
competition be of one of the forms described in Article (3).
programme contain an AGREEMENT and CONDITIONS OF CON-
architect and owner in conformity with those printed in the
s circular, that it include no provision at variance therewith,
erms of payments in accord with good practice, and that it spe-
the nature of expert engineering services for which the architect
ed.
programme make provision for a jury of at least three persons.
programme conform in all particulars to the spirit of this cir-

gramme meets the above essential conditions, the approval of
be given to it by the subcommittee for the territory in which
erected, or if there be no subcommittee for that territory, then
ommittee on competitions. If, for legal or other reasons, the
ee deem that deviations from the essential conditions are
ive the approval of the Institute to a programme containing
ower to give approval in such cases is, however, vested only
mmittee. The professional adviser, when duly authorized
proper committee, may print the Institute's approval as
ramme or otherwise communicate it to those invited to

of Agreement between Owner and Competitors

the submission of drawings in this competition (here insert
r or of the body duly authorized to enter into contracts
), hereinafter called the OWNER, agrees with the competi-
lly that the owner will, within .... days of the date
f drawings, make an award of the commission to design
forming the subject of this competition to one of those
drawings in consonance with the mandatory require-
, and will thereupon pay him, on account of his services
his total estimated fee as stated below. And further,
submission of drawings as aforesaid and the mutual
the subjoined CONDITIONS OF CONTRACT BETWEEN
the owner agrees and each competitor agrees, if the
r, immediately to enter into a contract containing all
ving, and until such contract is executed to be bound

connected with the making of proposals and contrac of payment, to keep proper accounts and generally administrative duties connected with the work.

(4) **Supervision.** The architect is to supervise th committed to his control.

ARTICLE II. DUTIES OF THE C

(1) **Payments.** The owner is to pay the archite equal to .... per cent * upon the cost of the work. of payments should be here stated.) †

(2) **Reimbursements.** The owner is to reimbur to time, the amount of expenses necessarily incurr while traveling in the discharge of duties connected w

(3) **Service of Engineers.** The owner is to reimb of the services of such engineers for heating, mechani are specifically provided for in each programme. The and their compensation shall be subject to the approv

(4) **Information, Clerk of the Works, etc.** Th formation as to his requirements; to pay for all neces tests, and for the continuous services of a clerk of the is approved by the architect.

Standard Form of Competition-Pro

The following standard form of COMPETITION-PROC American Institute of Architects, contains those prov considers essential to the fair and equitable conduct Institute in no way assumes or attempts to dictate a ducting a competition; it claims only the right to con having found by experience the danger to the inter COMPETITOR from a competition in which such prov

member to take part in any competition which does not meet those
l conditions, and the programme of which has not been specifically
d. A competition should be of such form as to establish equitable rela-
tween the Owner and the Competitor. To insure this, the require-
hould be clear and definite; the competency of the Competitors should
red; the agreement between the Owner and Competitors should be
as becomes a plain statement of business relations; and the judgment
e based on expert knowledge. The following programme will, if adhered
luly approved by the Institute Subcommittees on Competitions for
ious chapters of the Institute, and by the Standing Committee on
itions of the Institute.

Programme of Competition for

. . . .

(Insert name of proposed building)

Throughout this programme the word Owner is used to indicate either the
person, or those to whom he has delegated his powers.

PART I

roposed Building. The. . . . .
(Insert name of owner)

s to erect a new. . . . .
(Insert name of building)

site at. . . . .
(Insert location)

uthority. The. . . . .
(Insert name of owner)

legated to. . . . .
(Insert name or names of individuals)

ty to select an architect to prepare the plans for, and supervise the
of the building.

E. If authority for the erection of the proposed building is granted by act of
re, ordinance, etc., it is desirable to make clear the source of such authority.

rchitectural Adviser. The Owner has appointed as his expert Pro-
al Adviser . . . .
(Insert name and address of adviser)

are this programme and to act as his Adviser in the conduct of this
ition.

E. No competition shall be instituted without the aid of a competent adviser.
ld be an architect of the highest standing and his selection should be the Owner's
. He should be chosen with the greatest care, as the success of the competition
end largely upon his experience and ability. The duties of the expert are to
hose who hold the competition in regard to its form and terms, to draw up the
me, to advise in choosing the Competitors, to answer inquiries from Competitors
eneral to direct the competition.

Competitors. Participation in this competition is limited
the following architects:. . . . .
(Insert names of invited competitors)

(A) which will consist of the following members:.....

....

(Insert names of jury)

Or (B) which will consist of................members.

(Insert number)

has appointed the following:.........................

....

(Insert names of those so selected)

the COMPETITORS will select the remaining members of t

NOTE. To insure a just and wise award and to protect
OWNER and the COMPETITORS, the drawings should be subm
secure expert knowledge and freedom from personal bias. Th
least three members, one of whom must, and the majority of w
architects, for example, a layman and an architect selected by t
COMMITTEE, and an architect selected by the COMPETITORS.
tance it is desirable to increase the size of the JURY, adding to
qualified laymen. Some of the advantages of a JURY so constit
understands and can explain the intent of the drawings, and
author's skill in design, arrangement and construction. Becau
its judgment on the merits of the designs submitted is of the hi
The adoption of the recommendation that the architectural m
the majority, is not necessarily a cause of expense, for the reas
the proper conduct of competitions, many architects of standing
warrants, to serve as JURORS without payment, other than a
tomary and desirable that the COMPETITORS should elect one o
members of the JURY. It is not advisable that the PROFESS
drawn up the programme, be permitted to vote as a member
may with advantage take part in the deliberations of the JURY

(6) **Authority of Jury.** The OWNER agrees that the
selected as above provided, will have authority to make
decision in the matter shall be final. Moreover, this Ju
to one of those taking part in this competition, unless
which fulfils the mandatory requirements of this pro

he report of the Jury and will thereupon, without learning the identity of the Com-
ors, select as the winner of the competition the author of the design selected by
ry, unless in his judgment there be cause to depart from such selection, in which
e will, still without learning the identity of the Competitors, select one of the other
s submitted in competition. The Owner further agrees that he will pay to the
r of the design designated as most meritorious by the Jury, in case he should not
pointed Architect of the building, a prize of $.................................

(State amount of prize)

opening of the envelope containing the name of the author of the design selected
Owner will automatically close the contract between him and the Owner, printed
t III hereof.

Examination of Designs and Award. The Professional Adviser will
ine the designs to ascertain whether they comply with the mandatory re-
ments of the programme, and will report to the Jury any instance of failure
mply with these mandatory requirements. The Owner further agrees
he Jury will satisfy itself of the accuracy of the report of the Professional
ser, and will place out of competition and make no award to any design
does not comply with these mandatory requirements. The Jury will
lly study the programme and any modifications thereof, which may have
made through communications (see Section (12)), and will then consider the
ining designs, holding at least two sessions on separate days, and consider-
each session all the drawings in competition, and will make the award,
he classification of prize-winners, if prizes are given, by secret ballot, and
ajority vote, before opening the envelopes which contain the names of the
etitors. In making the award the Jury will thereby affirm that it has
no effort to learn the identity of the various Competitors, and that it has
ined in ignorance of such identity until after the award was made. The
ing of the envelope containing the name of the author of the selected design,
utomatically close the contract between him and the Owner, printed as
III hereof.

) **Report of the Jury.** The Jury will make a full report which will state
asons for the selection of the winning design and its reason for the classifica-
of the designs placed next in order of merit, and a copy of this report, accom-
d by the names of prize-winners, if prizes are given, will be sent by the
essional Adviser to each Competitor. Immediately upon the opening
e envelopes, the Professional Adviser will notify all Competitors, by
of the result of the competition.

) **Compensation to Competitors.** The Owner agrees to pay to the suc-
l competitor within ten days of the judgment, on account of his fee for
es as architect, one-tenth of his total estimated fee.

full discharge of his obligation to them (in case prizes or fees are offered),
wner agrees:

) To pay the following prizes to those ranked by the Jury next to the suc-
l design: To the design placed second $........., to the design placed
$........., to the design placed fourth $........., to the design placed
$.........., etc., within ten days of the judgment, or

) To pay to each of the Competitors invited to take part in this competi-
other than the successful Competitor, a fee of $........ within ten days
e judgment.

o) **Exhibition of Drawings.** It is agreed that no drawings shall be ex-
ted or made public until after the award of the Jury. There will be a public
bition of all drawings after the judgment, and all drawings, except those

of the successful competitor, will be returned to their authors at the clo
thereof.

(11) **Use of Features of Unsuccessful Designs.** Nothing original in the unsuccessful designs shall be used without consent of, or compensation to, the author of the design in which it appears. In case the Owner desires to make use of any individual feature of an unsuccessful design, the same may be obtained by adequate compensation to the designer, the amount of such compensation to be determined in consultation with the author and the Professional Adviser.

(12) **Communications.** (**Mandatory.**) If any Competitor desires infor-mation of any kind whatever in regard to the competition, or the programme, he shall ask for this information by anonymous letter addressed to the Pro-fessional Adviser, and in no other way, and a copy of this letter and the answer thereto will be sent simultaneously to each Competitor, but no re-quest received after ....
(Insert date)
will be answered.

(13) **Anonymity of Drawings.** (**Mandatory.**) The drawings to be sub-mitted shall bear no name or mark which could serve as a means of identification, nor shall any such name or mark appear upon the wrapper of the drawings, nor shall any Competitor directly or indirectly reveal the identity of his design or hold communication regarding the competition with the Owner or with any member of the Building Committee or of the Jury, or with the Professional Adviser, except as provided for under Communications. It is understood that in submitting a design, each Competitor thereby affirms that he has complied with the foregoing provisions in regard to anonymity and agrees that any vio-lation of them renders null and void this agreement and any agreement aris from it. With each set of drawings must be enclosed a plain, opaque, seal envelope without any superscription or mark of any kind, same containing the name and address of the Competitor. These envelopes shall be opened by the Professional Adviser after the final selection has been made, and prefera in the presence of the Jury.

(14) **Delivery of Drawings.** (**Mandatory.**) The drawings submitted in this competition shall be securely wrapped, addressed to the Professional Ad-viser at ....in plain lettering and
....
(Insert address for delivery of drawings)
with no other lettering thereon, and delivered at this address not later than
....
(Insert date and hour)

In case drawings are sent by express, they may be delivered to an express com-pany at the above date and hour, in which case the express company's receipt, bearing date and hour, shall be mailed immediately to the Professional Ad-viser as evidence of delivery.

PART II

(15) **Site.** The site of the building is as follows..
(Insert description of site, and provide topographical map giving dimensions, grades,

NOTE. The site should be carefully described and a survey of the property sh attached and included as part of the programme. Conditions pertaining to the s to neighboring buildings frequently become determining factors in a design. graphs showing surrounding buildings and landscape-conditions may with advan included.

. (**Mandatory.**) For the purpose of this competition the cost of
shall be figured at.........cts per cu ft, and the total thereof
(Insert number)
is basis shall not exceed..................................
(Insert limit of cost)

age. (**Mandatory.**) Cubage shall be so computed as to show as
ssible the actual volume of the building, calculated from the finished
s of the lowest floor to the highest points of the roofs, and contained
tside surfaces of the walls. Pilasters, cornices, balconies and other
ctions shall not be included. Porticos with engaged columns and
ctions shall be taken as solids and figured to the outer face of the
hen columns are free-standing, one-half of the volume of the porti-
aken. There shall also be included in the cubage the actual volume
ts, towers, lanterns, dormers, vaults, and other features adding to
the building, also the actual volume of exterior steps above grade.
f an area of less than 400 sq ft shall not be deducted. In calculat-
account shall be taken of variations in the exterior wall-surface, as
the projection of a basement-story beyond the general line of the
figured diagram showing method adopted in cubing shall accom-
et of drawings.

wings. (**Mandatory.**) The drawings submitted shall be made
the following list, at the scale given, and rendered as noted; and
wings than these shall be submitted:

....
(Insert list, scale and method of rendering)

he drawings submitted should be the least number necessary to set forth
lution of the problem, and the scale of these drawings the smallest com-
he requirement that the intention of each COMPETITOR be made clear to an
Where the number and scale of drawings is reduced to the minimum, and
ds of rendering imposed, the COMPETITORS are enabled to devote their time
the study of the problem, which is the serious business of a competition,
on draughtsmanship and rendering, which when carried beyond a certain
no value whatever in determining the fitness of the COMPETITORS to handle
recting the building, for which the competition is being held.

PART III

Agreement between Owner and Competitors

ration of the submission of drawings in this competition, and the
nises enumerated in the subjoined CONDITIONS OF CONTRACT BE-
ITECT AND OWNER the OWNER agrees, and each COMPETITOR agrees
be made in his favor, immediately to enter into a contract contain-
ONDITIONS here following, and until such contract is executed, to be
e said CONDITIONS.

nditions of Contract between Architect and Owner

Duties of the Architect

n. The ARCHITECT is to design the entire building and its imme-
ndings and is to design or direct the design of its constructive,
and decorative work and its fixed equipment and, if further re-
ovable furniture and the treatment of the remainder of its grounds.

(2) **Drawings and Specifications.** The ARCHITECT is to make such revisi
of his competitive scheme as may be necessary to complete the prelimina
studies; and he is to provide drawings and specifications necessary for the co
duct of the work. All such instruments of service are and remain the pr
erty of the ARCHITECT.

(3) **Administration.** The ARCHITECT is to prepare or advise as to all fo
connected with the making of proposals and contracts, to issue all certific
of payment, to keep proper accounts and generally to discharge the nece
administrative duties connected with the work.

(4) **Supervision.** The ARCHITECT is to supervise the execution of a
work committed to his control.

Duties of the Owner

(5) **Payments.** The OWNER is to pay the ARCHITECT for his ser
sum equal to . . . . .per cent upon the cost of the wprk.

NOTE. The percentage inserted should be in accord with good practice. T
and amounts of payments should be here stated. Good practice has establi
payments on account as follows: Upon completion of the preliminary studies
of the total estimated fee less the previous payment; upon completion of contract
and specifications two-fifths additional of such fee; for other drawings, for s
and for administration, the remainder of the fee, from time to time, as the work p

(6) **Reimbursements.** The OWNER is to reimburse the architect f
to time, the amount of expenses necessarily incurred by him or his
while traveling in the discharge of duties connected with the work.

(7) **Service of Engineers.** The OWNER is to reimburse the A
the cost of the services of ENGINEERS for. . . . .
(Insert nature of work for which the OWNER agrees that ENGINEERS shall be e
his expense)

The selection of such ENGINEERS and their compensation shall be
the approval of the OWNER.

(8) **Information, Clerk of the Works, Etc.** The OWNER is to
formation as to his requirements; to pay for all necessary surveys,
tests, and for the continuous services of a clerk of the works whose
is approved by the ARCHITECT.

PART IV

Requirements of the Building

NOTE. For the same reason that elaborate drawings are undesirabl
to avoid lengthy and detailed instructions as to the desired accomm
confuse the problem and hamper the COMPETITORS; and the OWNER
benefit he might gain in allowing the COMPETITORS freedom to devel
they would not otherwise be at liberty to suggest. It should be borne i
the cost of the building, as determined by its cubical contents, shou
requirements of the OWNER in regard to the design, materials of const
of rooms, etc., should be fixed, but not both. If, on the one hand, t
and cost is fixed, it should be stated that the requirements of the Ow
to as closely as possible by COMPETITORS; if, on the other hand, the
OWNER are definitely fixed, it may be stated that the cubical con
while not limited, will be taken into consideration in making the
sizes of certain rooms, etc., are definitely fixed, the word MANDAT
at the head of the paragraph referring to these rooms.

Here should follow a list of rooms required, together with
which apply to the building under consideration.

ANDARD DOCUMENTS OF THE AMERICAN INSTITUTE OF ARCHITECTS*

y **Notes.** This introductory paragraph is from an article† by R.
is, President of The American Institute of Architects. "For many
and owners have commonly used an agreement recognized as in-
mperfect, and one apt to lead to serious misunderstandings, if not
lties. Architects entrusted with important work and its accom-
ısibilities have endeavored to have agreements drawn which would
eguard the interests involved. When, some nine years ago (1907),
ttempted to prepare a new standard agreement, it found already
erable number of forms prepared by architects, differing in detail
ı one main point. This one point was that the contract and the
he contract should be treated as two branches of the same agree-
ne document, nor yet as two. The contract was to be as brief as
g simply what the obligation was. The conditions of the contract,
ıd involved, yet essential to the contract, were of necessity com-
thy. The most difficult part of the work, surveying the field and
he way, was done by the Committees on Contracts and Specifica-
he years 1906 to 1911, and resulted in the first edition of the
UMENTS, published in 1911. At that time some thought the prob-
hers thought it but an important step forward; which latter proved
These first documents, excellent as they were as text-books, were
r everyday use. The Institute again took up the problem, this
definite aim to produce a document which should entirely replace
greement when the contract for its publication expired in May,
as been done and the carefully studied AGREEMENT and CONDI-
CONTRACT presented to the convention in December, 1914, have
udied and improved and are now (1915) on the market for general
nal study between January and May, 1915, the Institute had the
coöperation with representatives of many of the building trades
e of counsel representing the Institute and counsel representing
rades. The document, like its predecessor, will now come to the
use. It will prove to be imperfect and revised sections will be
it is believed to be in the main a fair and comprehensive agree-
that is practical and fit for general use. Architects everywhere
se and test this form, and criticism from owners and builders will
ived and considered. In addition to this most important docu-
mittee has prepared and the Institute has published a form of
R OF ACCEPTANCE by a contractor of a sub-contractor's bid, and an
tween a contractor and sub-contractor. Many architects who have
which a bond has been required have been surprised at the ease
obligations of the bond could be evaded. In most cases, because
itect, contractor, or owner, had invalidated the bond. The new
is prepared for insuring, as far as possible, that the bonding com-
charge its obligations and protect the owner who pays for this
'he LETTER FROM CONTRACTOR TO SUB-CONTRACTOR is intended to
le form whereby the mutual obligations of the two shall be clearly
AGREEMENT BETWEEN CONTRACTOR AND SUB-CONTRACTOR accom-
ne purpose in a somewhat more formal way."

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l by permission of The American Institute of Architects.
l in the Journal of The American Institute of Architects, June, 1915.

chapters of the Institute and to engineers, contractors
the country. The documents were prepared under t
Kane, counsel for the Institute, and Ernest Eidlitz, an
criticism of Professor Samuel Williston of the Harvard
assistance of James W. Pryor, in their editing. The
to the work in 1911. The Standing Committee on C
during the preparation of the first edition of the STA
Grosvenor Atterbury, Chairman; Allen B. Pond, Sec
William A. Boring, Frank C. Baldwin, Frank W. F
G. L. Heins. Criticisms of the first edition of the D
the Institute and during the year 1913 a group of arch
ton, known as the Joint Committee of the Boston So
the Master Builders' Association, gave much sincere
the same time the National Association of Builders' E
criticism of the documents.

In 1914 the Institute instructed its Standing Com
Specifications to undertake a general revision with a
DITIONS simpler in wording and more equitable. The
to hold conferences with organizations so desiring. Su
tory of the several chapters of the Institute were appoi
the standing committee. The Boston group presented
entirely new draft which proved of high value and
Parker, was present with the Standing Committee a
The Committee had a joint meeting with representati
ciation of Builders' Exchanges and thereafter the co
W. B. King, and the counsel of the Institute, Louis
rated most effectively with the committee. The G
entirely rewritten and in response to the strong desire o
tractors, the principle of GENERAL ARBITRATION, subj
documents, was adopted, and provisions relative to th
TRACTOR AND HIS SUBCONTRACTORS were included in the
study, conference and criticism, a draft of the secon

ECIFICATIONS. It is, however, wise to assemble them in a single
nd, since they have as much bearing on the DRAWINGS as on the
NS, and even more on the business relations of the contracting
are properly called the GENERAL CONDITIONS OF THE CONTRACT.
EMENT, GENERAL CONDITIONS, DRAWINGS and SPECIFICATIONS are
ent elements of the CONTRACT and are acknowledged as such in the
they are correctly termed the CONTRACT DOCUMENTS. Statements
one of them are just as binding as if made in the AGREEMENT. The
orms, although intended for use in actual practice, should also be
a code of reference representing the judgment of the Institute as to
utes good practice and as such they may be drawn upon by archi-
roving their own forms. Although the forms are suited for use in
with a single or general contract, they are equally applicable to an
nducted under separate contracts.

the Standard Documents and Approval of Same. The new
CUMENTS of The American Institute of Architects are now on sale*
n office and drafting-supplies in all the large cities of the country,
he old UNIFORM CONTRACT which is to be discontinued. The follow-
itles of the STANDARD DOCUMENTS: A. I. FORM OF AGREEMENT AND
RAL CONDITIONS OF THE CONTRACT. B. BOND OF SURETYSHIP. C.
UBCONTRACT. D. LETTER OF ACCEPTANCE OF SUBCONTRACTOR'S
A cover in heavy paper with valuabe EXPLANATORY NOTES is sent
rge with each complete set of the documents or with each copy of
AGREEMENT and GENERAL CONDITIONS OF THE CONTRACT. These
have received the full approval of the Institute, through its con-
rd of directors and officers. They are the outcome of nine years of
work by a Standing Committee on Contracts and Specifications.
tee, comprising some of the ablest American architects, was assisted
tute's forty Chapters; advised by eminent legal specialists in con-
d aided by representatives of the Building and Trade Associations

Architects, Builders and Contractors. The CONTRACT FORMS may be
y or in lots from the usual dealers. If your dealer cannot supply you send
l his name to The Secretary, A. I. A., The Octagon, Washington, D. C. All
clude the necessary remittance irrespective of A. I. A. membership and irre-
mmercial standing of purchaser. The Institute has adopted these CASH
which no exception will be made to anybody, in order to reduce cost of
and thereby reduce expense to the user. Remittances may be by check,
cash, or stamps. All prices are f. o. b. Washington, D. C. If small lots are
parcel-post instead of express, add 1 ct for AGREEMENT and GENERAL CONDI-
ct apiece for each of the other forms according to quantity ordered; other-
will be made by express, collect.

Single Copies: Agreement and General Conditions in cover, \$0.10; Bond
\$0.02; Form of Subcontract, \$0.02; Letter of Acceptance of Subcontrac-
, \$0.02; Complete set in cover, \$0.15. A Trial set will be delivered upon
2-cent stamps.

Quantities and Discounts to Architects, Builders and Contractors.
antities are subject to the following discounts (which are also given by all

of 100 (one kind or assorted); 10% on lots of 500 (one kind or assorted); 15%
(one kind or assorted). As these DOCUMENTS are printed on large sheets and
ities, they cannot be supplied with any individual names or printing different
lard forms. The Institute does not wish to encourage the use of the AGREE-
eral conditions other than those endorsed by it, but on request will sell the
parate from the STANDARD GENERAL CONDITIONS at 2 cts each.

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ARCHITECTS, THE OCTAGON, WASHINGTON, D
IS TO BE USED ONLY WITH THE STANDA
CONDITIONS OF THE CONTRA

THIS AGREEMENT, made the ....
day of................in the year Nineteen Hundre
by and between....................(Two blank line
hereinafter called the Contractor, and...........(Tw
....her
WITNESSETH, that the Contractor and the Own
hereinafter named agree as follows:

Article 1. The Contractor agrees to provide all th
all the work shown on the Drawings and described in

(Here insert the caption descriptive of the work as used i
ditions, Specifications, and upon the Drawings.)

..................................(Five blank lines)......
prepared by.......................(Two blank lines)
acting as, and in these Contract Documents entitled
everything required by the General Conditions of th
tions and the Drawings.

Article 2. The Contractor agrees that the work u
substantially completed..........................

(Here insert the date or dates of completion, and stipu
damages if any.)

...............................(Eight blank lines).....

Article 3. The Owner agrees to pay the Contracto
performance of the Contract

....
....($....
to additions and deductions as provided in the Gener
tract and to make payments on account thereof as pr

(Here insert provisions as to the method and ti

d or herein repeated; and that the following is an exact enumera-
ecifications and Drawings:

...............(Thirty-five blank lines)...................................

ctor and the Owner for themselves, their successors, executors, and assigns, hereby agree to the full performance of the covenants ed.

S WHEREOF they have hereunto set their hands and seals, the first above written.

Owner's witnesses) .. } as to..... (Repeated for Owner)..... (SEAL)
.... }

IE GENERAL CONDITIONS OF THE CONTRACT [*]

ARD FORM OF THE AMERICAN INSTITUTE OF ARCHITECTS

been approved by the National Association of Builders' Exchanges, The ation of Master Plumbers, and the National Association of Master Steam Fitters.

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inciples and Definitions.

ntract Documents consist of the Agreement, the General Con- Contract, the Drawings and Specifications. These form the

ner, the Contractor and the Architect are those named as such in t. They are treated throughout the Contract Documents as if he singular number and masculine gender.

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blished by permission of The American Institute of Architects.

(c) The Contractor shall, as in Article 43, be responsible to tl
acts and omissions of his subcontractors and of all persons direc
employed by him or them in connection with the work.

(d) The term Subcontractor includes only those having a dire
the Contractor and it includes one who furnishes material even
no work.

(e) The term "person" or "anyone" as employed herein sl
include a firm or corporation.

(f) Written notice shall be deemed to have been duly serve
person to the individual or to a member of the firm or to an office
tion for whom it is intended, or if delivered at or mailed to t
address known to him who gives the notice.

(g) The term "work" of the Contractor or Subcontractor in
materials or both.

(h) When the words "approved," "satisfactory," "equal to,"
directed," etc., are used, approval, etc., by the Architect, is und

(j) All time-limits stated in the Contract Documents are of th
contract.

(k) The law of the place of building shall govern the const
contract.

Art. 2. Execution, Correlation and Intent of Document
tract Documents shall be signed in duplicate by the Owner and C
case of failure to sign the General Conditions, Drawings or Spe
Architect shall identify them. Even though the signatures of th
the Contractor may have been attested by witnesses they may be p
competent evidence. The Contract Documents are complementar
is called for by any one shall be as binding as if called for by all. T
of the documents is to include all labor and materials reasonably n
the proper execution of the work. It is not intended, however, th
or work not covered by or properly inferable from any heading, bra
trade of the specifications shall be supplied unless distinctly so noted o
ings. Materials or work described in words which so applied have a
technical or trade meaning shall be held to refer to such recognized

Art. 3. Detail Drawings and Instructions. The Architect sh
with reasonable promptness, additional instructions, by means of d
otherwise, necessary for the proper execution of the work. All suc
and instructions shall be consistent with the Contract Documents, tr
ments thereof, and reasonably inferable therefrom. The work shall
in conformity therewith and the Contractor shall do no work with
drawings and instructions. The Contractor and the Architect, if e
quests, shall jointly prepare a schedule, subject to change from tim
accordance with the progress of the work, fixing the latest dates a
various detail drawings will be required, and the Architect shall fur
accordance with that schedule. Under like conditions, a schedule
pared, fixing dates for the submission of shop drawings, for the
manufacture and installation of materials and for the completion o
parts of the work.

Art. 4. Copies Furnished. Unless otherwise provided in
Documents the Architect will furnish to the Contractor, free of cha
of drawings and specifications reasonably necessary for the executio

Art. 5. Shop Drawings. The Contractor shall submit two
shop or setting drawings and schedules required for the work of the

t shall pass upon them with reasonable promptness. The Con-
e any corrections required by the Architect, file with him two
and furnish such copies as may be needed. The Architect's
drawings or schedules shall not relieve the Contractor from
deviations from drawings or specifications, unless he has in
e Architect's attention to such deviations at the time of sub-
it relieve him from responsibility for errors of any sort in shop
dules.

ings and Specifications on the Work. The Contractor shall
f all drawings and specifications on the work, in good order,
rchitect and to his representatives.

rship of Drawings and Models. All drawings, specifications
f furnished by the Architect are his property. They are not to
work and, with the exception of the signed contract-set, are to
m on request, at the completion of the work. All models are
he Owner.

les. The Contractor shall furnish for approval all samples as
ork shall be in strict accordance with approved samples.

Architect's Status. The Architect shall have general super-
ion of the work. He is not the agent of the Owner, except as
ntract documents and when in special instances he is authorized
to act, and in such instances he shall, upon request, show the
en authority. He has authority to stop the work whenever
y be necessary to insure the proper execution of the Contract.
mination of the employment of the Architect, the Owner shall
and reputable Architect, whose status under the contract shall
mer Architect.

Architect's Decisions. The Architect shall, within a reason-
decisions on all claims of the Owner or Contractor and on all
ting to the execution and progress of the work or the interpre-
tract documents. Except as may be otherwise expressly pro-
ded to these General Conditions or as particularly set forth in
all the Architect's decisions are subject to arbitration.

man, Supervision. The Contractor shall keep on the work a
l foreman and any necessary assistants, all satisfactory to the
general foreman shall not be changed except with the consent
The foreman shall represent the Contractor in his absence
given to him shall be as binding as if given to the Contractor.
st such directions shall be confirmed in writing to the Contrac-
ctor shall give efficient supervision to the work, using his best
n. He shall carefully study and compare all drawings, specifi-
instructions and shall at once report to the Architect any error,
omission which he may discover.

erials, Labor, Appliances. Unless otherwise stipulated, the
rovide and pay for all materials, labor, water, tools, equipment,
necessary for the execution of the work. Unless otherwise
erials shall be new and both workmanship and materials shall
y. The Contractor shall, if required, furnish satisfactory evi-
nd and quality of materials. The Contractor shall not employ
unfit person or anyone not skilled in the work assigned to him.

ection of Work. The Owner, the Architect and their repre-
t all times have access to the work wherever it is in preparation

or progress and the Contractor shall provide proper facilities fo
for inspection. If the specifications, the Architect's instruct
nances or any public authority require any work to be specia
proved, the Contractor shall give the Architect timely notice o
inspection and the Architect shall promptly inspect it. If any
be covered up without approval or consent, it must, if required
be uncovered for examination at the Contractor's expense. R
questioned work may be ordered by the Architect and, if found
with the Contract, all expense of reexamination and replacemen
by the Contractor, otherwise it shall be allowed as extra work.

Art. 14. Correction of Work Before Final Payment.
shall promptly remove from the premises all materials, whethe
worked, and take down and remove all portions of the work co
Architect as failing to conform to the Contract; and the C
promptly replace and re-execute his own work in accordance wi
and without expense to the Owner and shall bear the expense of
work of other contractors destroyed or damaged by such removal
If the Contractor does not remove such condemned work and ma
reasonable time, fixed by written notice, the Owner may remove
store the material at the expense of the Contractor. If the Cont
pay the expense of such removal within five days thereafter, the Ow
ten-days' written notice, sell such materials at auction or at private
account for the net proceeds thereof, after deducting all the costs
that should have been borne by the Contractor.

Art. 15. Deductions for Uncorrected Work. If the Archit
inexpedient to correct work injured or not done in accordance with t
the difference in value together with a fair allowance for damage
ducted, if acceptable to the Owner.

Art. 16. Correction of Work After Final Payment. Neither t
tificate nor payment nor any provision in the Contract Documents s
the Contractor of responsibility for negligence or faulty materials or
ship within the extent and period provided by law, and upon writte
shall remedy any defects due thereto and pay for any damage to oth
sulting therefrom. All questions arising under this Article shall be dec
Articles 10 and 45.

Art. 17. Protection of Work and Property. The Contractor
tinuously maintain adequate protection of all his work from damage
protect the Owner's and adjacent property from injury arising in
with this Contract. He shall make good any such damage or injury,
as may be directly due to errors in the contract documents.

Art. 18. Emergencies. In an emergency affecting the safety of l
structure or of adjoining property, not considered by the Contracto
the provisions of Article 17, then the Contractor, without special ins
authorization from the Architect or Owner, is hereby permitted to
discretion, to prevent such threatened loss or injury and he shall so
appeal, if so instructed or authorized. Any compensation claimed
him therefor shall be determined under Articles 10 and 45 regardless
tations in Article 25 and in the second paragraph of Article 24.

Art. 19. Damage to Persons. In addition to the liability im
upon the Contractor on account of bodily injury or death suffered
Contractor's negligence, which liability is not impaired or oth
hereby, the Contractor hereby assumes, in cases not embraced

igation to save the owner harmless and indemnify him from
ability or payment (voluntary payments excepted), by reason
any person or persons, including death, suffered through any act
ne Contractor or any Subcontractor, or anyone directly or in-
d by either of them, in the prosecution of any work included in

bility Insurance. The Contractor shall maintain such insur-
ct him from claims under workmen's compensation acts and from
for damages for personal injury, including death, which may
tions under this contract. Certificates of such insurance shall
Owner, if he so require, and shall be subject to his approval for
ection. The Owner shall be responsible for his own contingent

Insurance. The Owner shall effect and maintain fire insur-
tire structure on which the work of this contract is to be done
erials, tools and appliances in or adjacent thereto and intended
to at least eighty per cent of the insurable value thereof. The
be made adjustable with and payable to the Owner as Trustee
concern. All policies shall be open to inspection by the Con-
Owner fails to show them on request or if he fails to effect or
ice as above, the Contractor may insure his own interest and
hereof to the Owner. If the Contractor is damaged by failure
maintain such insurance, he may recover under Art. 39. If
ig by any party in interest, the Owner as Trustee shall, upon the
s, give bond for the proper performance of his duties. He shall
ey received from insurance in an account separate from all his
he shall distribute it in accordance with such agreement as the
t may reach, or under an award of arbitrators appointed, one by
ier by joint action of the other parties in interest, all other pro-
accordance with Art. 45. If after loss no special agreement is
nt of injured work shall be ordered under Art. 24. The Trustee
to adjust and settle any loss with the insurers unless one of the
ested shall object in writing within three working days of the
s and thereupon arbitrators shall be chosen as above. The
hat case make settlement with the insurers in accordance with
such arbitrators, who shall also, if distribution by arbitration is
uch distribution.

aranty Bonds. The Owner shall have the right to require
o give bond covering the faithful performance of the contract
of all obligations arising thereunder, in such form as the Owner
d with such sureties as he may approve. If such bond is re-
tions given previous to the receipt of bids, the premium shall be
ractor; if subsequent thereto, it shall be paid by the Owner.

h Allowances. The Contractor shall include in the contract
es named in the Contract Documents and shall cause the work
done by such contractors and for such sums as the Architect may
act sum being adjusted in conformity therewith. The Con-
g up his bid, shall add such sums for expenses and profit on
llowances, as he deems proper, and no demand for expenses or
those included in the contract sum shall be allowed. The Con-
required to employ for any such work a Subcontractor against
sonable objection.

(b) By Unit Prices named in the contract or sub
(c) By Cost and Percentage or by Cost and a fix
(d) If none of the above methods is agreed upon,
receive an order in writing signed by the Owner and
tect, shall proceed with the work, no appeal to arb
such order to proceed.

In cases (c) and (d), the Contractor shall keep and
Architect may direct, a correct account of the net
together with vouchers. In any case, the Architect
including a reasonable profit, due to the Contractor.
tion of value, payments on account of changes shall
certificate.

Art. 25. Claims for Extras. If the Contracto
tions, by drawings or otherwise, involve extra cost
give the Architect written notice thereof before pro
and, in any event, within two weeks of receiving suc
cedure shall then be as provided in the last parag
claim shall be valid unless so made.

Art. 26. Applications for Payments. The Co
Architect an application for each payment and, if
vouchers from Subcontractors showing his payment
labor as required by Article 44. If payments are
done, such application shall be submitted at least te
falls due. If required, the Contractor shall before
to the Architect a schedule of values of the various
ing the total sum of the contract, divided so as to
contractors in accordance with Article 44 (e) made ou
tect may direct and, if required, supported by evi
This schedule, when approved by the Architect, sha
tificates of payment, unless it be found to be in e
ments, the Contractor shall submit a statement base
required, itemized in such form as the Architect may

he specifications, and of all claims by the Contractor, except those
de and still unsettled. Should the Owner fail to pay the sum
certificate of the Architect or in any award by arbitration, upon
due, the Contractor shall receive, in addition to the sum named
e, interest thereon at the legal rate in force at the place of building.

ayments Withheld. The Architect may withhold or, on account
discovered evidence, nullify the whole or a part of any certificate
protect the Owner from loss on account of:
e work not remedied.
iled or reasonable evidence indicating probable filing of claims.
f the Contractor to make payments properly to subcontractors
or labor.
able doubt that the contract can be completed for the balance

above grounds are removed certificates shall at once be issued
ithheld because of them.

ens. Neither the final payment nor any part of the retained
ll become due until the Contractor, if required, shall deliver to
mplete release of all liens arising out of this contract, or receipts
ereof and, if required in either case, an affidavit that the releases
clude all the labor and material for which a lien might be filed;
ctor may, if any subcontractor refuses to furnish a release or re-
rnish a bond satisfactory to the Owner, to indemnify him against
ien or otherwise. If any lien or claim remain unsatisfied after all
made, the Contractor shall refund to the Owner all moneys that
be compelled to pay in discharging such lien or claim, including
reasonable attorney's fee.

ermits and Regulations. The Contractor shall obtain and pay
and licenses, but not permanent easements, and shall give all
fees, and comply with all laws, ordinances, rules and regulations
work. If the drawings and specifications are at variance there-
tractor shall notify the Architect in writing before the work is
the value of any necessary changes shall be adjusted under Art.
the Contractor's work shall be done contrary to such laws, ordi-
nd regulations, without such notice, he shall bear all costs arising

oyalties and Patents. The Contractor shall pay all royalties
s and shall defend all suits or claims whatsoever for infringement
rights and shall save the Owner harmless from loss on account

se of Premises. The Contractor shall confine his apparatus,
materials and the operations of his workmen to limits indicated by
s, permits, or directions of the Architect and shall not encumber
ith his materials. The Contractor shall not load or permit any
ucture to be loaded with a weight that will endanger its safety.
r shall enforce the Architect's instructions regarding signs, ad-
ires and smoking.

leaning Up. The Contractor shall at all times keep the premises
mulations of waste material or rubbish caused by his employees
t the completion of the work he shall remove all his rubbish from
building and all his tools, scaffolding and surplus materials, and
work clean and *ready for use. In case of dispute the Owner may*

remove the rubbish and charge the cost to the several contractors as the Architect shall determine to be just.

Art. 34. Cutting, Patching and Digging. The Contractor shall do al cutting, fitting, or patching of his work that may be required to make its several parts come together properly and fit it to receive or be received by work of other contractors shown upon, or reasonably implied by, the Drawings and Specifica tions for the completed structure, and he shall make good after them, as the Architect may direct. Any cost caused by defective or ill-timed work shall be borne by the party responsible therefor. The Contractor shall not endanger any work by cutting, digging, or otherwise and shall not cut or alter the work of any other contractor, save with the consent of the Architect.

Art. 35. Delays. If the Contractor is delayed in the completion of the work by any act or neglect of the Owner or the Architect, or of any employee of either, or by any other contractor employed by the Owner, or by changes ordered in the work, or by strikes, lockouts, fire, unavoidable casualties, or any causes beyond the Contractor's control, or by delay authorized by the Architect pending arbi tration, or by any cause which the Architect shall decide to justify the delay, then the time of completion shall be extended for such reasonable time as the Architect may decide. No such extension shall be made for delay occurring more than seven days before claim therefor is made in writing to the Architect. In the case of a continuing cause of delay, only one claim is necessary. If no schedule is made under Art. 3, no claim for delay shall be allowed on account of failure to furnish drawings until two weeks after demand for such drawings and not then unless such claim be reasonable.

Art. 36. Owner's Right to Do Work. If the Contractor should neglect to prosecute the work properly or fail to perform any provision of this contract, the Owner, after three-days' written notice to the Contractor, may, without prejudice to any other remedy he may have, make good such deficiencies and may deduct the cost thereof from the payment then or thereafter due the Con tractor; provided, however, that the Architect shall approve both such action and the amount charged to the Contractor.

Art. 37. Owner's Right to Terminate Contract. If the Contractor shoul be adjudged a bankrupt, or if he should make a general assignment for the ben of his creditors, or if a receiver should be appointed on account of his insolv or if he should, except in cases recited in Article 35, persistently or repeated refuse or fail to supply enough properly skilled workmen or proper materials, if he should fail to make prompt payment to subcontractors or for materia labor, or persistently disregard laws, ordinances or the instructions of the Ar tect, or otherwise be guilty of a substantial violation of any provision of the tract, then the Owner, upon the certificate of the Architect that sufficient c exists to justify such action, may, without prejudice to any other right or re and after giving the Contractor seven-days' written notice, terminate th ployment of the Contractor and take possession of the premises and of al terials, tools and appliances thereon and finish the work by whatever met may deem expedient. In such case the Contractor shall not be entitled ceive any further payment until the work is finished. If the unpaid bal the contract price shall exceed the expense of finishing the work, includi pensation to the Architect for his additional services, such excess shall to the Contractor. If such expense shall exceed such unpaid balance, tractor shall pay the difference to the Owner. The expense incurre Owner as herein provided, and the damage incurred through the C default, shall be certified by the Architect.

38. **Contractor's Right to Stop Work or Terminate Contract.** If
should be stopped under an order of any court, for a period of three
through no act or fault of the Contractor or of anyone employed by him,
Owner should fail to pay to the Contractor, within seven days of its
and presentation, any sum certified by the Architect or awarded by
rs, then the Contractor may, upon three-days' written notice to the
nd the Architect, stop work or terminate this contract and recover from
er payment for all work executed and any loss sustained upon any plant
ial and reasonable profit and damages.

39. **Damages.** If either party to this contract should suffer damage by
otherwise, except as provided in Art. 40, because of any act or neglect
her party or of anyone employed by him, then he shall be reimbursed
ther party for such damage. Claims under this clause shall be made in
o the party liable within a reasonable time of the first observance of such
and not later than the time of final payment, except in case of claims
rticle 16, and shall be adjusted by agreement or arbitration.

40. **Mutual Responsibility of Contractors.** Should the Contractor
1 (c)) cause damage to any other person (see Art. 1 (e)) employed on the
e Contractor agrees, upon due notice, to settle with such person by agree-
arbitration, if such person will so settle. If such person sues the Owner
nt of any damage alleged to have been so sustained, the Owner shall
he Contractor, who shall, at his own expense, defend such proceedings
ny judgment against the Owner arise therefrom, the Contractor shall
atisfy it and pay all costs incurred by the Owner. The Contractor, if
l by any person held to the Owner by stipulations such as the above,
settle with such person by agreement or arbitration and in no case to
Owner on account of such damage.

41. **Separate Contracts.** The Owner reserves the right to let other
s in connection with this work. The Contractor shall afford other con-
reasonable opportunity for the introduction and storage of their ma-
nd the execution of their work and shall properly connect and coordinate
with theirs. If any part of the Contractor's work depends for proper
n or results upon the work of any other contractor, the Contractor shall
nd promptly report to the Architect any defects in such work that render
able for such proper execution and results. His failure so to inspect and
hall constitute an acceptance of the other contractor's work as fit and
or the reception of his work, except as to defects which may develop in
r contractor's work after the execution of his work. To insure the
xecution of his subsequent work the Contractor shall measure work
in place and shall at once report to the Architect any discrepancy be-
e executed work and the drawings.

42. **Assignment.** Neither party to the Contract shall assign the con-
thout the written consent of the other, nor shall the Contractor assign
eys due or to become due to him hereunder, without the previous written
of the Owner.

43. **Subcontracts.** The Contractor shall notify the Architect in writ-
e names of subcontractors proposed for the principal parts of the work
such others as the Architect may direct and shall not employ any that
itect may within a reasonable time object to as incompetent or unfit.
tractor may *in his discretion or shall, if so required,* submit with his pro-
ist of subcontractors. If the change of any name on such list is re-
permitted after signature of agreement, the contract price shall be

specifically noted to the contrary in a subcontract appr
quate by the Owner or Architect.

The Subcontractor agrees:

(a) To be bound to the Contractor by the terms of t
Drawings and Specifications and to assume toward him
responsibilities that he, by those documents, assumes to

(b) To submit to the Contractor applications for paym
time as to enable the Contractor to apply for payment
General Conditions.

(c) To make all claims for extras, for extensions of tim
delays or otherwise, to the Contractor in the manner p
Conditions for like claims by the Contractor upon the
time for making claims for extra cost as under Article 25
tions is one week.

The Contractor agrees:

(d) To be bound to the Subcontractor by all the oblig
assumes to the Contractor under the General Conditions,
cations and by all the provisions thereof affording reme
Contractor from the Owner.

(e) To pay the Subcontractor, upon the issuance of cert
the schedule of values described in Article 26 of the G
amount allowed to the Contractor on account of the Subc
extent of the Subcontractor's interest therein.

(f) To pay the Subcontractor, upon the issuance of cert
wise than as in (e), so that at all times his total payme
proportion to the value of the work done by him as the to
the Contractor is to the value of the work done by him.

(g) To pay the Subcontractor to such extent as may be
tract Documents or the subcontract, if either of these
larger payments than the above.

Contractor to the Subcontractor during the first ten days of the calendar
following that in which the claim originated.
To give the Subcontractor an opportunity to be present and to submit
e in any arbitration involving his rights.
To name as arbitrator under Article 45 of the General Conditions the per-
minated by the Subcontractor, if the sole cause of dispute is the work,
ls, rights, or responsibilities of the Subcontractor; or, if of the Sub-
tor and any other subcontractor jointly, to name as such arbitrator the
upon whom they agree.
Contractor and the Subcontractor agree that:
In the matter of arbitration, their rights and obligations and all procedure
e analogous to those set forth in Article 45 of the General Conditions.
ing in this Article shall create any obligation on the part of the Owner to
or to see to the payment of any sums to any Subcontractor.

45. Arbitration. Subject to the provisions of Article 10, all questions
ute under this contract shall be submitted to arbitration at the choice of
party to the dispute. The general procedure shall conform to the laws of
te in which the work lies and wherever permitted by law the decision of
itrators may be filed in court to carry it into effect. The demand for
tion shall be filed in writing with the Architect, in the case of an appeal
is decision, within ten days of its receipt and in any other case within a
ble time after cause thereof and in no case later than the time of final
nt, except as to questions arising under Article 16. If the Architect fails
e a decision within a reasonable time, an appeal to arbitration may be
as if his decision had been rendered against the party appealing. The
may agree upon one arbitrator; otherwise there shall be three, one named
ing by each party and the third chosen by these two arbitrators or, if they
select a third within ten days he shall be chosen by the presiding officer of
arest Bar Association. Should the party demanding arbitration fail to
n arbitrator within ten days of his demand, his right to arbitration shall
Should the other party fail to choose an arbitrator within such ten days,
chitect shall appoint such arbitrator. Should either party refuse or
to supply the arbitrators with any papers or information demanded in
, the arbitrators are empowered by both parties to take *ex-parte* proceed-
The arbitrators shall act with promptness. The decision of any two shall
ding on all parties to the dispute. The decision of the arbitrators upon
estion subject to arbitration under this contract shall be a condition
nt to any right of legal action. The arbitrators, if they deem that the
mands it, are authorized to award to the party whose contention is sus-
such sums as they shall deem proper for the time, expense and trouble
t to the appeal and, if the appeal was taken without reasonable cause,
s for delay. The arbitrators shall fix their own compensation, unless
ise provided by agreement and shall assess the costs and charges of the
tion upon either or both parties. The award of the arbitrators must be
ng and, if in writing, shall not be open to objection on account of the form
proceedings or the award.

hereinafter called the Principal, and. . . . .

(Here insert the name and address or legal title of one o

. . . . (Two blank lines) . . . .

. . . . (Two blank lines) . . . .

hereinafter called the Surety or Sureties are held and firml

(Here insert the name and address or legal title of

. . . . (Two blank lines). . . . .

hereinafter called the Owner, in the sum of. . . . .

. . . . (Two blank lines) . . . .

for the payment whereof the Principal and the Surety or Su their heirs, executors, administrators, successors and assi ally, firmly, by these presents.

Whereas, the Principal has, by means of a written Agre . . . . .entered into a contr . . . . Two blank lines) . . . . a copy of which Agreement is by reference made a part

Now, Therefore, the Condition of this Obligation is suc shall faithfully perform the Contract on his part, and demands, incurred for the same, and shall fully indemnify Owner from all cost and damage which he may suffer by do, and shall fully reimburse and repay the Owner all out the Owner may incur in making good any such default, sons who have contracts directly with the Principal for l this obligation shall be null and void; otherwise it shall r effect.

Provided, however, that no suit, action or proceeding by whatever shall be brought on this Bond after. . . . . the day on which the final payment under the Contract

ıed and Sealed this..............day of..........................19..
:sence of

.... } ....(SEAL)
(Repeated three times) } as to (Repeated three times)
.... } ....

C. THE STANDARD FORM OF SUBCONTRACT *

R USE IN CONNECTION WITH THE GENERAL CONDITIONS OF THE CONTRACT AS ISSUED BY THE AMERICAN INSTITUTE OF ARCHITECTS

form has been approved by the National Association of Builders' Exchanges, The ıal Association of Master Plumbers, and the National Association of Master Steam ot Water Fitters.

COPYRIGHT 1915 BY THE AMERICAN INSTITUTE OF ARCHITECTS, THE OCTAGON, WASHINGTON, D. C.

IS AGREEMENT, made this..................day of.............19..
d between..hereinafter called
ıbcontractor and..
ıafter called the Contractor.

TNESSETH, That the Subcontractor and Contractor for the considerahereinafter named agree as follows:

:tion 1. The Subcontractor agrees to furnish all material and perform all as described in Section 2 hereof for....(Here name the kind of building.)....
..............................(Blank lines).................................
..................(Here insert the name of the Owner.)........................
..............................(Blank lines).................................
ıafter called the Owner, at.......(Here insert the location of the work.)......
..............................(Blank lines).................................
:ordance with the General Conditions of the Contract between the Owner he Contractor, and in accordance with the Drawings and the Specifications red by.........................hereinafter called the Architect, all of General Conditions, Drawings and Specifications signed by the parties to or identified by the Architect, form a part of a Contract between the Conır and the Owner dated..................19.. and hereby become a part of Contract.

ction 2. The Subcontractor and the Contractor agree that the materials furnished and work to be done by the Subcontractor are (Here insert a e description of the work, preferably by reference to the numbers of the Drawings he pages of the Specifications.)..
..............................(Blank lines).................................

ction 3. The Subcontractor agrees to complete the several portions and 'hole of the work herein sublet by the time or times following:............ Here insert the date or dates and if there be liquidated damages state them.).....
..............................(Blank lines).................................

ction 4. The Contractor agrees to pay the Subcontractor for the performof his work the sum of..........(Blank line)............($..........) rrent funds, subject to additions and deductions for changes as may be d upon, and to make payments on account thereof in accordance with on 5 hereof.

ction 5. The *Contractor and Subcontractor agree to* be bound by the *of the General Conditions, Drawings and Specifications* as far as applicable

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* *Published by permission of The American Institute of Architects.*

to this subcontract, including the provisions of Article
ditions of the Contract, as follows*:

Section 6...

..............................(One page of blank lines).....

Finally. The Subcontractor and Contractor, for t
successors, executors, administrators and assigns, do h
performance of the covenants herein contained.

IN WITNESS WHEREOF they have hereunto set t
date first above written.

In Presence of

....

....

D. STANDARD FORM OF ACCEPTANCE TRACTOR'S PROPOSAL†

FOR USE IN CONNECTION WITH THE STANDARD DOCUM
CAN INSTITUTE OF ARCHITECTS

This form has been approved by the National Association of
National Association of Master Plumbers, and the National As
and Hot Water Fitters.

COPYRIGHT 1915 BY THE AMERICAN INSTITUTE OF
OCTAGON, WASHINGTON, D. C.

Dear Sir: Having entered into a contract with (Here
dress or corporate title of the Owner.)........................
....................................(Blank line)..........
for the erection of (Here insert the kind of work and the
erected.)..
....................................(Blank line)..........
in accordance with plans and specifications prepared by
and address of the Architect.)..........................
....................................(Blank line)..........
and in accordance with the General Conditions of the C
specifications, the undersigned hereby accepts your propo
....
to provide all the materials and do all the work of (Her
to be done, as plumbing, roofing, etc., accurately describing b
drawings and specifications governing such work.)..........
....................................(Blank lines)..........

The Undersigned agrees to pay you in current funds fo
ance of the subcontract established by this acceptance of

Subcontractor entering into this agreement should be sure that not merely
ove Article 44, but the full text of the General Conditions of the Contract
ied by the Owner and Contractor is known to him, since such full text,
i not herein repeated, is binding on him.

RCHITECTS' LICENSE LAW.* STATE OF ILLINOIS

Act to Provide for the Licensing of Architects and Regulating the Practice of Architecture as a Profession

Appointment of a State Board of Examiners of Architects

TION 1. Be it enacted by the People of the State of Illinois, represented
ieral Assembly. That within thirty days after the passage of this act
vernor of this state shall, by the advice and consent of the Senate, appoint
e Board of Examiners of Architects, to be composed of five members,
whom shall be a member of the faculty of the Illinois State University,
e other four shall be architects residing in the State of Illinois, who have
ngaged in the practice of architecture at least ten years. Two of the said
ing architects appointed as examiners shall be designated to hold office
years from the date of the passage of this act, and the other two, together
ie member of the faculty aforesaid, shall hold office for four years from the
e of this act; and thereafter upon the expiration of the term of office of
rson so appointed, the Governor of the state shall appoint a successor to
erson whose term of office shall expire, to hold office for four years, and
rson so appointed shall have the above specified qualifications. In case
tment of a successor is not made before the expiration of the term of any
er, such member shall hold office until his successor is appointed and duly
ed. Any vacancy occurring in membership of the board shall be filled
Governor of the state for the unexpired term of such membership. [Sec-
and 3 relate to the organization of the board, salaries, etc.]

minations. Applicants for License to Pay an Examination Fee of $15 and a License Fee of $25

TION 4. Provisions shall be made by the board hereby constituted for
g examinations at least twice in each year, of applicants for license to
e architecture, and any person over twenty-one years of age, upon pay-
of a fee of fifteen dollars ($15) to the secretary of the board, shall be
d to an examination for determining his or her qualifications. All exami-
s shall be made directly by said board, or a committee of two members
ted by the board, and due notice of the time and place of the holding of
xaminations shall be published, as in the case provided for the publication
rules and regulations thereof. The examination shall have special refer-
the construction of buildings, and a test of the knowledge of the candidate
strength of materials and of his or her ability to make practical applica-
f such knowledge in the ordinary professional work of an architect, and in
ities of a supervisor of mechanical work on buildings, and should also seek
ermine his or her knowledge of the laws of sanitation as applied to build-
If the result of the examination of any applicant shall be satisfactory to

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nacted by the Fortieth General Assembly at the Regular Biennial Session, Ap-
June 3, 1897, and In Force July 1, 1897; *with Amendments* Adopted by the
first General Assembly and Approved April 19, 1899; and by the Forty-fourth
Assembly, Approved May 16, 1905, and by the Forty-seventh General Assembly,
d May 26, 1911.

the State Board of Examiners of Architects that he or
practice of the profession of architecture on the date of
shall be entitled to a license without an examination, pr
shall be made within six months after passage of this a
granted, shall set forth the fact that the person to who
was practicing architecture in this state at the time of
and is therefore entitled to a license to practice architectu
tion by the board of examiners, and the secretary of th
payment to him of the fee of twenty-five dollars ($25), is
in said affidavit, a license to practice architecture in t
with the provisions of this act. In the case of a copartne
member whose name appears must be licensed to pra
stock-company or corporation shall be licensed to pr
the same may employ licensed architects. Each licens
his or her license recorded in the office of the county c
county in this state in which the holder thereof shall prac
pay to the clerk the same fee that is charged for the rec
missions. A failure to have his or her license so rec
sufficient cause for revocation of such license.

County Clerks to Keep Record of Licenses

SECTION 6. Each county clerk shall keep in a book, pr
a complete list of all licenses recorded by him under the
together with the date of the issuance of each license.

Licensed Architects to Have a Se

SECTION 7. Every licensed architect shall have a s
which must contain the name of the architect, his or her
the words, "Licensed Architect," "State of Illinois," wit
all drawings and specifications issued from his office for

n to practice architecture without a license in this state, or to advertise,
t any sign or card, or other device which might indicate to the public
t she is entitled to practice as an architect.*

Persons Who are to be Regarded as Architects

N 9. Any person who shall be engaged in the planning or supervision
ection, enlargement, or alteration of buildings for others, and to be
ed by other persons than himself, shall be regarded as an architect
e provisions of this act, and shall be held to comply with the same;
ng contained in this act shall prevent the draughtsmen, students, clerks
or superintendents, and other employees of those lawfully practicing
cts, under license as herein provided for, from acting under the instruc-
rol or supervision of their employers; or shall prevent the employment
ntendents of buildings paid by the owners from acting, if under the
nd direction of a licensed architect who has prepared the drawing and
ons for the building. The term building in this act shall be understood
ructure, consisting of foundations, walls, and roof, with or without the
ts; but nothing contained in this act shall be construed to prevent any
echanic, or builder from making plans and specifications for, or super-
erection, enlargement or alteration of any building that is to be con-
by himself or employees; nor shall a civil engineer be considered as an
unless he plans, designs and supervises the erection of buildings, in
e he shall be subject to all the provisions of this act, and be considered
hitect.

License Revoked

N 10. Architects' license issued in accordance with the provisions of
hall remain in full force until revoked for cause, as hereinafter provided.
se so granted may be revoked by unanimous vote of the State Board
ners of Architects for gross incompetency, or recklessness in the con-
of buildings, or for dishonest practices on the part of the holder thereof;
e any license shall be revoked such holder shall be entitled to at least
ays' notice of the charge against him, and of the time and place of the
f the board for the hearing and determining of such charge. And on
lation of such license it shall be the duty of the secretary of the board
tice of such cancelation to the county clerk of each county in the state
the license has been recorded, whereupon the clerks of the counties
k the license recorded in his office "canceled." After the expiration
nths from the revocation of a license, the person whose license was
nay have a new license issued to him by the secretary upon certificate
ard of Examiners, issued by them upon satisfactory evidence of proper
r his reinstatement, and upon payment to the secretary of the fee of
rs ($5).

e purpose of carrying out the provisions of this Act relating to the
n of licenses, the board and each member thereof shall have the power
ister oaths, and said board shall have the power to secure by its sub-
h the attendance and testimony of witnesses, and the production of
l papers relevant to any investigations by the board for the purpose of
out the provisions of this Act, relating to the revocation of licenses.
shall be entitled to the same fees as witnesses in a court of record to
n like manner. The accused shall be enltited to the subpœna of the
his witnesses, and to be heard in person or by counsel in open public
y circuit court of this state or any judge thereof, either in term-time

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* As amended *May 16, 1905.*

the practice of his profession shall annually, during the
in such practice, pay to the secretary of the board duri
fee of five dollars ($5), and the secretary shall thereupo
architect a certificate of renewal of his license for the t
licensed architect who shall fail to have his license renew
July in each and every year shall have his license revok
duty of the secretary of the board to give notice of such r
clerk in each county in the state, whereupon the clerk
make an entry of such revocation accordingly. But th
license in apt time shall not deprive such architect of the
after; and the secretary of the board shall give like n
but the fee to be paid upon the renewal of license after t
be ten dollars ($10), to cover the additional expense inc
account of such notices.†

Report of Proceedings to be Filed with the Auditor

SECTION 12. Within the first week of December, after
board, and annually thereafter, the secretary of the bo
auditor of the state a full report of the proceedings of the
affidavits of the president and secretary, subject to the
auditor.

EDUCATIONAL INSTITUTIONS IN THE AND CANADA OFFERING COURSES TECTURE. TRAVELING-FELLO AND SCHOLARSHIPS.‡

Academy of Architecture and Industrial Science,
Maack, Principal. This is a private school founded by
and designed more particularly to meet the wants of bui

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* As amended May 26, 1911. This amendment to the secon
defines the authority of the board in conducting the trials of a

em such instruction as is necessary to attain the highest proficiency in
trade and a thorough understanding of the plans and details of compli-
buildings. There is also a special course for those desiring to fit themselves
sitions as draughtsmen in architects' offices. Tuition for the regular
is $50 for a three-month term, or $300 for the full course of eight terms,
o for the year. There are several special courses which may be commenced
time and for which the tuition varies.

erican Academy in Rome, Fellowship in Architecture. ROMAN
. The fellowship is awarded annually and is of the value of $1 000 a year
ree years. The award is made on competitions which are open only to
rried male citizens of the United States, who comply with the regulations
Academy. Candidates are required to be (1) graduates of one of the
ectural schools included in the accepted list of the Academy; or (2) grad-
of a college or university of high standing who hold certificates of at least
ears' study in one of such architectural schools; or (3) Americans who
pils of the first class of the School of Fine Arts at Paris, and who have
ed at least three values in that class. There is no age-limit. Information
the terms and conditions of the competitions may be obtained from the
ary of the Academy, 101 Park Avenue, New York City.

erican School of Correspondence, Chicago, Ill. Correspondence-
s in Architecture, Architectural Engineering, Contracting and Building,
orced Concrete, Architectural Design, and Structural Draughting. Bulle-
nt on application.

nour Institute of Technology, Chicago, Ill. Walter Francis Shattuck,
sor of Architecture. Full four-year course leading to the degree of
lor of Science in Architecture. Applicants for admission must have com-
the regular four-year high-school course. A HOME TRAVELING SCHOLAR-
four prizes and a medal are awarded annually. Tuition $175 per year.

aux-Arts Architects, Society of. Wendell P. Blagden, Chairman, Com-
on Education, 437 Fifth Avenue, New York City. The course established
93 now consists of a series of thirty-five competitions for the study of
ectural design and the styles of architecture, open to draughtsmen and
nts in architectural schools in the United States and Canada, and modeled
system of instruction adopted by the École des Beaux Arts in Paris. The
is free, except for the annual fee of $2 for registration of each student.
are no restrictions as to the age, nationality or sex of the students. No
inary examinations are given but new students are expected to have a
ledge of the five orders of architecture. Bronze medals are presented for
ence in design and money-prizes are offered in special prizes for decoration,
-planning of buildings, etc. The competitions are conducted by the
nittee on Education. The PARIS PRIZE is conducted by the Annual Paris
Committee. (See following paragraph.) The students of the Society of
Arts Architects are classified according to the number of "values"
ed in Class B and Class A competitions from the time of commencing the
. Certificates are presented to all students of Class A completing the
as defined in the circular of information, which will be furnished on request
chairman of the committee on education. During the season of 1913–1914
ork was carried on by ninety-six local representatives of the committee on
tion in seventy-eight different cities and over one thousand students were
red.

PARIS PRIZE OF THE SOCIETY OF BEAUX ARTS ARCHITECTS. Conducted
e Annual Paris Prize *Committee, William Emerson, Chairman,* 281 Fifth

Avenue, New York City. A scholarship offered to the winner of the final
petition, who fulfills the requirements as stated in the circular of inform
concerning the Paris Prize, which will be furnished on request by the chai
of the Paris Prize committee. The Paris Prize winner is authorized by
of the French Minister of Public Instruction and Fine Arts to follow the le
and take part in the competitions of the first class in the section of archite
subject to the approval of the faculty of the École Nationale et Spécial
Beaux Arts, Paris. The winner of the prize will receive $250 quarterly fo
and one-half years and the four unsuccessful competitors in the final compe
may receive $100 each, provided the committee considers their work satisfa
The competition will consist of two preliminary and one final competition.
Paris Prize competitions are open to all citizens of the United States
twenty-seven years of age, on July 1, of the current season, independent
connection with the Society of Beaux Arts Architects.

Carnegie Institute of Technology, Pittsburgh, Pa. School of Ar
Design. Henry Hornbostel, Patron. Department of Architecture.
McGoodwin, Professor in charge. (1) A complete course in architectu
day-students for which the degree of Bachelor of Arts is awarded to thos
cializing in design and allied subjects (Option 1) and the degree of Bache
Science to those specializing in construction and allied subjects. From fo
five years are required for the completion of prescribed work. (2) For gra
day-students, a course of advanced studies in design and allied subjects, sche
to cover one year, and leading to the degree of Master of Arts. (3) A p
day-course, scheduled to cover two years, for experienced draughtsmen a
signers, for which a certificate of proficiency is awarded. (4) A course for
students, for which a certificate of proficiency is awarded. This course in
the same work as is required of day-students in design, freehand drawin
history of architecture. Total tuition and incidental fees: for day-stu
residents of Pittsburgh, $43; for other day-students, $53; for night-stu
residents of Pittsburgh, $16; for other night-students, $18.

Columbia University, New York City. School of Architec
(1) Full four-year course leading to the degree of Bachelor of Archite
(2) Full four-year course leading to the Professional Certificate in Archite
(3) Advanced courses leading to the degree of Master of Arts and Doc
Philosophy. (4) Students not candidates for a degree or certificate are adn
under the Department of Extension Teaching and to the summer session wi
examination, and select their own programme of study. Tuition, $250 per
except for Extension Teaching, in which the average charge is $5 per le
hour per term. There are three Traveling Fellowships, the McKim Fel
ship and the Columbia Fellowship, awarded annually in May, and the Pe
Fellowship, awarded quadrennially in May, all open, by competition, to g
ates of the school within the preceding three years and to students who
fulfilled all requirements for the thesis. Each of these requires the winn
devote one year to foreign travel and study. Details in the Announce
of the School of Architecture, Columbia University.

Cornell University, Ithaca, N. Y. College of Architecture.
fessor Clarence A. Martin, Dean and Director. Professor Albert C. Ph
Secretary. (1) Courses leading to the degree of Bachelor of Archite
first, a four-year course covering the general field of architecture in his
theory, design, construction and practice; second, the same course modifi
the substitution of engineering and construction for a part of the adva
work in design and drawing; third, five-year courses covering the same gr
as the four-year courses but including additional work in the arts and sci

two-year special course in architecture, leading to a certificate. (3) Grad- courses in architecture and construction leading to the degree of Master chitecture. Tuition, $150 a year.

exel Institute of Art, Science and Industry, Philadelphia, Pa. SE IN ARCHITECTURE. Arthur Truscott, Professor of Architecture; John ill, Professor of Design. Three-year course in Architecture, a large share e time being devoted to purely architectural work. Tuition, $100 per year.

eorgia School of Technology, Atlanta, Ga. DEPARTMENT OF ARCHI- URE. Francis Palmer Smith, Professor in charge. (1) Full four-year course ng to the degree of Bachelor of Science in Architecture. (2) Two-year al course leading to a certificate of proficiency. Tuition, $25 per year esidents of Georgia; $100 for non-residents. The Atlanta Chapter of the rican Institute of Architects has provided a loan-fund in this department ne or two students needing pecuniary assistance.

arvard University, Graduate School of Applied Science, Cambridge, s. SCHOOL OF ARCHITECTURE. Herbert Langford Warren, Nelson Robin- Jr., Professor of Architecture, in charge. Eugene Duquesne, Professor of itectural Design. (1) Professional training in architecture, open to gradu- of colleges and scientific schools of good standing, leading to the degree laster in Architecture. Length of period of study, commonly three years, nding on ability and previous training. (2) Competent special students, must be over twenty-one years of age and must have had at least three s office-experience, admitted to any courses which their training fits them ursue with advantage. Tuition, $150 per year. (3) Advanced course in tectural design open to graduates in architecture from recognized pro- nal schools or to others of similar competence in Design. Such students also be candidates for the degree of Master in Architecture. Tuition in rdance with amount of work taken.

HOOL OF LANDSCAPE-ARCHITECTURE. Professor James Sturgis Pray, in ge. (1) Professional training in landscape architecture open to graduates lleges and scientific schools of good standing, leading to the degree of Mas- n Landscape-Architecture. (2) Competent special students admitted to courses for which their previous training fits them. Tuition, $150 per

vo TRAVELING-FELLOWSHIPS, the JULIA AMORY APPLETON and the ROBIN- are offered for competition in alternate years, each having an annual value 000, tenable for two years, for travel and study in Europe under the tion of the Department of Architecture. These Fellowships are open for petition to graduates in architecture of Harvard University.

SIDENT-SCHOLARSHIPS. Two AUSTIN SCHOLARSHIPS IN ARCHITECTURE one in LANDSCAPE-ARCHITECTURE, annual value, $300. ARCHITECTURAL UE SCHOLARSHIP IN ARCHITECTURE, open for competition to members of rchitectural League of America, three; annual value, $150. Other resident larships open to schools of architecture and of landscape-architecture in non with students in the graduate schools. Awarded by committee on ination of testimonials.

he International Correspondence Schools, Scranton, Pa. A corporation ed to furnish instruction by correspondence and to hold examinations to lish proficiency. The architectural course is designed particularly to meet wants of those already engaged in the building trades or drafting-room. cludes sixty-one subjects covering the elements of building-construction, ury, carpentry, plumbing, etc., and the principles of design, drawing, ren-

letters from former employers, together with drawings co
as fully as possible. All special students must take in thei
at the Institute courses in descriptive geometry and mech
these subjects have been passed at the September exam
standing, or excuse from one or both has been obtained on
work accomplished elsewhere. Tuition, $250 per year. An
Fellowship amounting to $1 000 is given solely on the
merit, candidates being received from both regular and sp
prizes, varying from $10 to $200 each, are equally divide
and the special students. Certain funds are available for
qualified regular students for undergraduate and for post

McGill University, Montreal, Canada. Departmen
Ramsay Traquair, Percy E. Nobbs and Thomas W. Ludlow,
(1) Full five-year course leading to the degree of Bach
(2) Competent special students are admitted to take a p
university certificate is granted for this work. Tuition,
George Creeford Browne Traveling-Scholarship
the value of $500, is awarded annually.

North Dakota Agricultural College, Fargo, N.
Engineering. E. S. Keene, Dean. Contractors' and bu
years; Certificate of Completion given. Full four-year c
leading to Bachelor of Science in Architecture. Full four-
tectural Engineering, leading to Bachelor of Science in Archi
Tuition free. Fees amounting to $25 per year.

Ohio Mechanics' Institute, Cincinnati, Ohio.
Arts. Department of Architecture. Henry Norton
plete course in architecture covering six years, divided into t
The instruction is arranged to unite the high-school and
uates of grammar-schools are trained in draughting and ele
subjects simultaneously with their high-school subjects du
The second period completes the full technical course,
mathematics and sciences, and leads to the degree of

:. Incidental fees amount to about $30 per semester, these fees
college fees. No course in architectural design.

itute, Brooklyn, N. Y. Course in Architecture. Walter Scott
or of the SCHOOL OF FINE AND APPLIED ARTS. (1) Two-year
itectural design. (2) Two-year course in architectural construc-
ll three-year course in architectural design and architectural con-
ne course in architectural design aims to give students a general
will prepare them to pursue the profession of architecture as com-
nts in architects' offices, and leads to positions of responsibility
ence. The course in architectural construction aims to fit the
neral draughting in builders' offices, or for general detailing and
work in an architect's office, and leads to the position of super-
construction-work. Tuition, $64 per year.

tute, Houston, Texas. ARCHITECTURAL DEPARTMENT. Wm.
, in charge. Full four-year course leading to Bachelor of Science
re. Tuition free.

echnic Institute, Terre Haute, Ind. DEPARTMENT OF ARCHI-
alverd A. Howe, Director. Full four-year course, designed to give
aining in architectural engineering together with systematic in-
rchitectural design. Tuition and incidental fees, $110.

veling-Scholarship, Inc. C. H. Blackall, Secretary, 20 Beacon
, Mass. Candidates must be under thirty years of age at the date
ing of the preliminary examinations. At that date they must
gaged in professional work during two years in Massachusetts in
f a practicing architect resident in Massachusetts, and will be
ss preliminary examinations upon the following subjects: (1) His-
ecture; (2) Freehand drawing from the cast; (3) Construction,
ractice; (4) An elementary knowledge of the French language.
legree in Architecture from the Massachusetts Institute of Tech-
nbia University, University of Pennsylvania, Cornell University,
ersity, or University of Illinois will be allowed to present such
n will be accepted in lieu of the examinations in the preliminaries.
ho pass in these preliminary examinations are admitted to a com-
sign, the successful candidate in which is awarded the scholarship
nnually, for two years, $1 000 to be expended in foreign travel and
Boston Society of Architects, through a committee, has complete
examinations and supervises the work of the scholar. The Society
awards the sum of $75 as a second prize.

University, Syracuse, N. Y., College of Fine Arts. DEPART-
HITECTURE. Frederick W. Revels, Director. (1) A full four-year
hitecture. (2) A full four-year course in architecture allowing
in architectural engineering. These courses lead to the degree of
rchitecture, (B.Ar.). (3) A two-year special course for architec-
smen of two or more years' office-experience, leading to a certifi-
ency. Tuition, $120 per year.

ne University of Louisiana, New Orleans, La. DEPARTMENT OF
E IN THE COLLEGE OF TECHNOLOGY. W. H. P. Creighton, Dean;
Professor of Architecture, in charge. (1) Full four-year course
egree in architecture. (2) Full four-year course leading to a degree
al engineering. (3) Special courses for students not candidates
Tuition, $100 per year. Special attention given to subtropical

University of California, Berkeley, Cal. DEPARTMENT OF ARCHITEC-
TURE. (1) Full four-year course leading to the degree of Bachelor of Scienc
(2) Two-year graduate course leading to advanced degrees. (3) Special o
elective courses for students not candidates for a degree. Tuition free to res
dents of the state of California.

University of Illinois, Urbana, Ill. COURSES IN ARCHITECTURE. L. I
Provine, Professor in charge. (1) Full four-year course leading to the degre
Bachelor of Science in Architecture. (2) Full four-year course, leading to
degree of Bachelor of Science in Architectural Engineering. Tuition is
Incidental fee, $24 per year. PLYM TRAVELING-FELLOWSHIP, $1 000 fo
year of travel abroad; awarded by competition to graduates of the Depar
of Architecture of the University of Illinois.

University of Michigan, Ann Arbor, Mich. DEPARTMENT OF
TECTURE, Emil Lorch, Professor of Architecture in charge of Dep
(1) A general four-year course leading to the degree of Bachelor of Arch
(2) A four-year course, in which architectural design is emphasized, l
the same degree. (3) A four-year course in which there is a large pro
engineering subjects, leading to the degree of Bachelor of Architectural
ing. (4) Five-year courses leading to the degrees of Master of Archi
Master of Architectural Engineering. (5) A two-year course for spec
(experienced draughtsmen or college graduates). (6) Students m
Bachelor-of-Arts degree and the degree in Architecture in from five
There are THREE SCHOLARSHIPS. Annual fees, $55 for students fr
and $65 for others.

University of Minnesota, Minneapolis, Minn. COURSES
TURE. Frederick Maynard Mann, Professor in charge. (1)
course, leading to the degree of Bachelor of Science in Architec
four-year course leading to degree of Bachelor of Science in Arch
neering. (3) Fifth year in each, leading to degree of Master
Special students of maturity and practical experience admitte
Incidental fee for residents and non-residents alike, $50 per y

University of Notre Dame, Notre Dame, Ind. COLLE
TURE. Rev. John Cavanaugh, President. Francis Wynne K
(1) Full four-year course in design leading to the degree of B
in Architecture. (2) Full four-year course in architectural
to the degree of Bachelor of Science in Architectural Eng
year special course leading to a Certificate of Proficiency. I
and tuition, $400 per year.

University of Oregon, Eugene, Oregon. SCHOOL
Ellis F. Lawrence, Director. Four-year course leadin
the profession of architecture. Two options in third a
(1) Architectural design. (2) Structural design. A
Arts is given for the successful completion of the first a
of Science for the second.

University of Pennsylvania, Philadelphia, Pa.
TURE. Warren Powers Laird, Professor in charge.
leading to the degree of Bachelor of Science in Ar
special course leading to a professional certificate.
one year leading to the degree of Master of Science

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* This information was taken from the Preliminary A
Architecture of the University of Oregon. The organizat
pleted (1915).

angement of courses in Arts and Architecture leading to the degree of r of Arts at the end of the fourth year and to the degree of Bachelor of in Architecture at the end of the sixth year. (5) Course in Architectural ring leading to the degree of Bachelor of Science in Architecture. Sum- ool, providing instruction in many architectural subjects of the regular PRIZES AND SCHOLARSHIPS are awarded annually. The PRIZE OF THE AN ACADEMY AT ROME and the PARIS PRIZE are open to students and the DSON SCHOLARSHIP is available to those resident in Pennsylvania. The and certificate are accepted by the American Institute of Architects of examination for membership. Instruction-staff (1914–15) in archi- 29 persons; students, 301, including summer-school courses. Tuition, er year. Circular and year-book on application to Professor John Dean of the Towne Scientific School, University of Pennsylvania, lphia, Pa.

JOHN STEWARDSON MEMORIAL SCHOLARSHIP IN ARCHITECTURE. The this scholarship, established in 1897, is $1 000. The holder is required d one year in travel and the study of Architecture in Europe under the n of the managing committee. All candidates must be under thirty age and either students or practitioners of architecture, resident in the Pennsylvania for at least one year immediately preceding the date of nary examinations.

ersity of Southern California. COLLEGE OF FINE ARTS. Four-year in architecture, leading to the degree of Bachelor of Fine Arts; also a -course of three years qualifying the student for practical work in archi- design.

ersity of Texas, Austin, Texas. SCHOOL OF ARCHITECTURE. F. E. e, Professor in charge. Four-year and five-year courses leading respec- o the degrees of Bachelor of Science and Master of Science in Architec- n the first, second and third years the course in prescribed; in the fourth h years the student has opportunity of selecting his studies so as to spe- n the æsthetic or in the engineering branches of architecture. There is ge for tuition, but a matriculation fee of $10 per year for the first three required.

ersity of Toronto, Toronto, Ontario, Canada. DEPARTMENT OF AR- TURE. C. H. C. Wright, Professor in charge. Full four-year course to the degree of Bachelor of Applied Science (B.A.Sc.) with an option of tural engineering replacing architectural design in the fourth year. The , first year, $100; second year, $110; third and fourth years, $120. The ity is supported by the Province of Ontario.

ersity of Washington, Seattle, Washington. COURSE IN ARCHITEC- Karl Gould, Professor in charge. A course in home-architecture and ion. Lectures and laboratory work.

hington, The State College of, Pullman, Wash. DEPARTMENT OF ECTURE. Rudolph Weaver, Professor in charge. (1) Full four-year eading to the degree of Bachelor of Science in Architecture. (2) Two- ecial course leading to a Certificate of Proficiency. (3) Special students, ely prepared, are admitted to all classes. Tuition free.

hington University, St. Louis, Mo. Course in Architecture. John y Robinson, Professor in charge. (1) Four-year course in architecture hitectural engineering leading to the degree of Bachelor of Science. (2) r course leading to the degree of Master of Science. (3) Special course loma. Tuition, $150 per year. Special course, $40 per year.

Wentworth Institute, Boston, Mass. A. L. Williston, Principal. Courses
in architectural construction and carpentry and building. (1) Two-year course
in architectural construction trains for positions of foremen, superintendents,
detail-designers, etc. Tuition, $18 per year and $15 laboratory-fee. (2) One
year course in carpentry and building planned for those wishing to enter the
woodworking-trades and industries. Tuition, $18 per year and $9 laboratory-
fee.

Yale University, New Haven, Conn. Courses in Architecture. Richa
Henry Dana, Professor in charge. Regular course covers three years. Spec
degree, Bachelor of Fine Arts, to be competed for at end of course. The fi
year's work, including lectures on history of chief styles of architecture a
principles of composition and practice in elementary design, may be taken as
elective by juniors and seniors in the academic course. Alice Kimball Engl
Scholarship supported from fund of $11 000 for a year's travel abroad. W
liam Wirt Winchester Scholarship, supported from fund of $20 000 f
year's travel abroad. Tuition, $180 per year.

ARCHITECTURAL SOCIETIES AND ORGANIZATIO OF THE WORLD *

1. United States

(1) THE AMERICAN INSTITUTE OF ARCHITECTS

The Octagon, Washington, D. C

List of Chapters (1915) of The American Institute of Arch

The year indicates the date of the chapter's organization

Baltimore Chapter. 1870
Boston Chapter. 1870
Brooklyn Chapter. 1894
Buffalo Chapter. 1890
Central New York Chapter. 1887
Cincinnati Chapter. 1870
Cleveland Chapter. 1890
Colorado Chapter. 1892
Columbus (Ohio) Chapter. 1913
Connecticut Chapter. 1902
Dayton Chapter. 1899
Georgia Chapter. 1906
Illinois Chapter. 1869
Indiana Chapter. 1910
Iowa Chapter. 1903
Kansas City Chapter. 1890
Louisiana Chapter. 1910
Louisville Chapter. 1908
Michigan Chapter. 1887
Minnesota Chapter. 1892
New Jersey Chapter. 1900
New York Chapter. 1867
North Carolina Chapter. 19
Oregon Chapter. 1911
Philadelphia Chapter. 1869
Pittsburgh Chapter. 1891
Rhode Island Chapter. 187
St. Louis Chapter. 1890
San Francisco Chapter. 18
South Carolina Chapter.
Southern California Chapte
Southern Pennsylvania Cha
Texas Chapter. 1913
Washington (D. C.) Chap
Washington State Chapte
Wisconsin Chapter. 191
Worcester Chapter. 189

List of State Associations of The American Institute

Pennsylvania State Association. 1909.
New York State Association. 1913

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and Specification-Index, published by The William T. Comstock Co

) ARCHITECTURAL LEAGUE OF AMERICA

ORGANIZATIONS OF THE LEAGUE

ociety of Washington University, St. Louis
ctural Club
ty of Cornell University
gton University Architectural Club, Washington, D. C.
, Philadelphia
itectural Club
ectural Club
hitectural Club
ectural Club
tectural Club
. C., Architectural Club
b, University of Illinois
arvard
ty of Mural Painters
Architectural Club
ctural Club

(3) MISCELLANEOUS SOCIETIES

ty of Landscape Architects
ociation of Indianapolis
Club of Minneapolis
eague, Pacific Coast
eague of New York
ociety of the University of California
itectural Club
ciety of Architects
ctural Club
of Architects
tute of Arts and Sciences
ects' Business Association
ectural Club
ation of Architects
hitectural Club
itectural Club
ety of Architects
ctural Club
ctural Club
Architects' Club of Louisville, Ky.
tion of Architects
of St. Paul
ectural Association
rchitectural Club
rchitects' Association
chitectural Club
Institute of Technology Architectural Association
rchitectural Club
ciety of Architects
rchitectural Club
ety of Architects
of Architects

Society of Architects of Akron, Ohio
Society of Architects of Columbia University
Society of Beaux-Arts Architects
Society of Naval Architects and Marine Engineers
South Bend Architectural Club
South Carolina Association of Architects
Southern States Engineering Society
Spokane Architectural Club
T Square Club of Philadelphia
Tacoma Society of Architects
Texas State Association of Architects
Utah Association of Architects
Washington, D. C., Architectural Club

2. Argentine Republic

Sociedad Central de Arquitectos. Buenos Aires

3. Austria

Austrian Society of Civil Engineers and Architects. V
Architekten-Klub der Weiner Kunstlergenossenschaft.
Gesellschaft Österreichischer Architekten. Vienna
Weiner Bauhütte. Vienna
Towarzystwo Politechniczne we Lwowie. Leopol
Towarzystwo Technisczne we Krakowie. Cracow

4. Belgium

Association des Architectes, de Liège. Liège
Société Centrale D'Architecture de Belgique. Brussels

6. Canada

ARCHITECTURAL ASSOCIATION OF CANADA

l Architectural Institute of Canada. Montreal
ta Association of Architects. Calgary and Edmonton, Alta.
tects' Association of Victoria. Victoria, B. C.
h Columbia Association of Architects.
ry Architectural Club
toba Association of Architects. Winnipeg, Man.
io Association of Architects. Toronto
nce of Quebec Association of Architects. Montreal
a Architectural Association. Regina, Sask.
tchewan Association of Architects. Regina, Sask.

7. Cuba

y of Engineers and Architects of Havana. Havana

8. France

anent Committee of International Congresses of Architects. Paris
é des Architectes Diplômés par le Gouvernement. Paris.
é Nationale des Architectes de France. Paris.
é Centrale des Architectes Français. Paris
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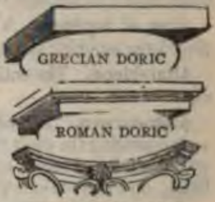
GLOSSARY *

al Terms, Ancient and Modern, Used by Architects, Builders, and Draughtsmen

's-Rod. An ornamental figure representing a rod with a serpent twined . It is sometimes confounded with the caduceus of Mercury. The on between the caduceus and the Aaron's-rod is that the former has two twined in opposite directions, while the latter has but one.

s. The upper member of the capital of a column. It is sometimes nd somtimes curved, forming on the plan s of a circle called the arch of the abacus, and only decorated with a rose or other ornament enter, having the angles, called horns of the cut off in the direction of the radius or curve. Tuscan or Doric, it is a square tablet; in c, the edges are molded; in the Corinthian, are concave and frequently enriched with In Gothic pillars it has a great variety of



ABACUS

y. A term for the church and other build- d by conventual bodies presided over by an abbess, in contradistinction to cathedral, which is presided over by a and priory, the head of which was a prior or prioress.

nent. That part of a pier from which the arch springs.

als. The boundings of a piece of land on other land, street, river, etc.

hus. A plant found in the south of Europe, representations of whose e employed for decorating the Corinthian and te capitals. The leaves of the acanthus are the bell of the capitol, and distinguish the orders from the three others.

eria. The small pedestals placed on the ies and apex of a pediment. They are usu- hout bases or plinths, and were originally to receive statues.

ACANTHUS

Aisle. The wings; inward side porticos of a church; the inward lateral which enclose the choir, the presbytery, and the body of the church sides. Any one of the passages in a church or hall into which the pews open.

e. The original and strict meaning of this word, which is derived from ish *alcoba*, is confined to that part of a bed-chamber in which the bed eparated from the other parts of the room by columns or pilasters. It mmonly used to express any large recess in a room, generally separated ch.

rion. In ancient Roman architecture, a room used by bathers for themselves.

lossary was compiled by Mr. Kidder from various sources, and with the some changes in typographical details to make it conform generally to the rest of the book it is left as published in the preceding editions.

Almonry. The place or chamber where alms were distributed to the churches, or other ecclesiastical buildings. At Bishopstone Church, Wi England, it is a sort of covered porch attached to the south transept, communicating with the interior of the church. At Worcester Cathedra land, the alms are said to have been distributed on stone tables, on ea within the great porch. In large monastic establishments, as at Westm it seems to have been a separate building of some importance, either join gate-house or near it, that the establishment might be disturbed as possible.

Altar. In ancient Roman architecture, a place on which offerings o fices were made to the gods. In Protestant churches, the communion often designated as the Altar, and in Roman Catholic churches it is a table placed at the east end of the church for the celebration of mass.

Altar of Incense. A small table covered with plates of gold on wh placed the smoking censer in the temple at Jerusalem.

Altar-piece. The entire decorations of an altar; a painting placed be altar.

Altar-screen. The back of the altar from which the canopy was sus and separating the choir from the lady chapel and presbytery. The Alta was generally of stone, and composed of the richest tabernacle work of finials, and pedestals, supporting statues of the tutelary saints.

Alto-rilievo. High relief. A sculpture, the figures of which project f surface on which they are carved.

Ambo. A raised platform, a pulpit, a reading-desk, a marble pulp oblong enclosure in ancient churches, resembling in its uses and positi modern choir.

Ambry. A cupboard or closet, frequently found near the altar in churches to hold sacred utensils.

Ambulatory. An alley — a gallery — a cloister.

Amphiprostylos. A Grecian temple which has a columned portico ends.

Amphitheater. A double theater, of an elliptical form on the plan, exhibition of the ancient gladiatorial fights and other shows. Its arena o which those exhibitions took place, was encompassed with seats risin each other, and the exterior had the accommodation of porticos or arc the public.

Amphora. A Grecian vase with two handles, often seen on medals.

Ancones. The consoles or ornaments cut on the key-stones of arche the sides of door-cases. They are sometimes made use of to support other figures.

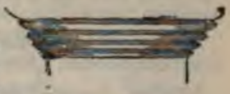
Angle-bar. In joinery, an upright bar at the angles of polygonal w a mullion.

Angle-capital. In Greek architecture, those Ionic capitals placed on t columns of a portico, which have one of their volutes placed horizontal angle of a hundred and thirty-five degrees with the plane of the frieze.

Annulated Columns. Columns clustered together by rings or bands used in English architecture.

Annular Vault. A vault rising from two parallel walls—the corridor. Same as Barrel Vault.

ulet. A small square molding used to separate others. The fillet which
tes the flutings of columns is sometimes known
s term.



ANNULET

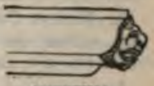
a, Antæ. A name given to a pilaster when
ed to a wall. Vitruvius calls pilasters *par-*
when insulated. They are not usually di-
ied, and in all Greek examples their capitals are
nt from those of the columns they accompany.

echamber. An apartment preceded by a vestibule and from which is
ched another room.

echapel. A small chapel forming the entrance to another. There are
les at Merton College, Oxford, and at King's College, Cambridge, England,
s several others. The antechapel to the lady-chapel in cathedrals is
lly called the Presbytery.

echoir. The part under the rood loft, between the doors of the choir
e outer entrance of the screen, forming a sort of lobby. It is also called
re-choir.

efixa. In classical architecture (gargoyles, in Gothic architecture), the
ents of lions' and other heads below the eaves of a
e, through channels in which, usually by the mouth, the
is carried from the eaves. By some this term is ap-
to the upright ornaments above the eaves in ancient
ecture, which hid the ends of the Harmi or joint tiles.



ANTEFIXA

phyge. The lowest part of the shaft of an Ionic or Corinthian column,
highest member of its base if the column be considered as a whole. The
yge is the inverted cavetto or concave sweep, on the upper edge of which
minishing shaft rests.

on. A plain or molded piece of finish below the stool of a window, put
cover the rough edge of the plastering.

e. The semicircular or polygonal termination to the chancel of a church.

eral. A temple without columns on the flanks or sides.

educt. An artificial canal for the conveyance of water, either above or
ground. The Roman aqueducts are mostly of the former construction.

besque. A building after the manner of the Arabs. Ornaments used by
me people, in which no human or animal figures appear.
sque is sometimes improperly used to denote a species of or-
ts composed of capricious fantastics and imaginary repre-
ons of animals and foliage so much employed by the Romans
decorations of walls and ceilings.

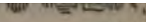
bian Architecture. A style of architecture the rudiments
ch appear to have been taken from surrounding nations, the
ians, Syrians, Chaldeans, and Persians. The best preserved
ens partake chiefly of the Græco-Roman, Byzantine, and
ian. It is supposed that they constructed many of their finest
gs from the ruins of ancient cities.

ostyle. That style of building in which the columns are
from one another from four to five diameters. Strictly
ng, the term should *be limited to intercolumniation* of four
ers, which is only suited to the Tuscan order.

ARABESQUE

ystylos. *That style of building in which four* columns
in the space of eight diameters and a half; the central

resist pressure.



ARCAD

Arch-buttress. Sometimes called a flying buttress; an arch springing from a buttress or pier.

Architrave. That part of an entablature which rests upon th column, and is beneath the frieze.

Architrave Cornice. An entablature consisting of an archit nice, without the intervention of the frieze, sometimes introduce venient to give the entablature the usual height.

Architrave of a Door. The finished work surrounding the upper part of the lintel is called the traverse; and the sides, the

Archives. A repository or closet for the preservation of writi

Archivolt. A collection of members forming the inner conto or a band or frame adorned with moldings running over the fac stones, and bearing upon the imposts.

Area. The superficial contents of any figure; an open space o a building; also, an uncovered space surrounding the foundation light to the basement.

Arena. The plain space in the middle of the amphitheater or public resort.

Arris. The meeting of two surfaces producing an angle.

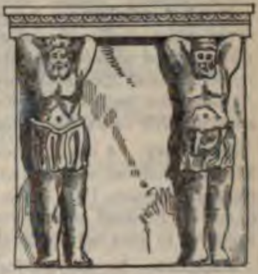
Arsenal. A public storehouse for arms and ammunition.

Artificer, or **Artisan.** A person who works with his hands, tures any commodity in iron, brass, wood, etc.

Ashlar, or **Ashler.** A facing made of squared stones, or a f thin slabs, used to cover walls of brick or rubble. *Coursed ashla* stones run in level courses all around the building; *random ash* stones are of different heights, but level beds. Common freest size, as they come from the quarry, are also called ashlar.

Asphaltum. A kind of bituminous stone, principally found i

Atlases, or **Atlantes**. Figures or half-figures of men, used instead of col-ans or pilasters to support an entablature; lled also Telamones.



ATLANTES

Atrium. A court in the interior division of oman houses.

Attached Columns. Those which project ree-fourths of their diameter from the wall.

Attic. A low story above an entablature, or ove a cornice which limits the height of the ain part of an elevation. Although the term is idently derived from the Greek, we find noth- g exactly answering to it in Greek architec- ure; but it is very common in both Roman and talian practice. What are otherwise called holobates in St. Peter's and St. Paul's Cathedrals are frequently termed attics.

Attic Order. A term used to denote the low pilasters employed in the decoration of an attic story.

Attributes. In painting and sculpture, symbols given to figures and statues to indicate their office and character.

Auditory. In ancient churches, that part of the church where the people usually stood to be instructed in the Gospel, now called the nave.

Aula. A court or hall in ancient Roman houses.

Aviary. A large apartment for breeding birds.

Axis. The spindle or center of any rotative motion. In a sphere, an imaginary line through the center.

Back-choir. A place behind the altar in the principal choir, in which there is, or was, a small altar standing back to back with the former.

Backing of a Rafter or Rib. The forming of an upper or outer surface, that it may range with the edges of the ribs or rafters on either side.

Backing of a Wall. The rough inner face of a wall; earth deposited behind a retaining wall, etc.

Back of a Window. That piece of wainscoting which is between the bottom of the sash frame and the floor.

Balcony. A projection from the face of a wall, supported by columns or consoles, and usually surrounded by a balustrade.

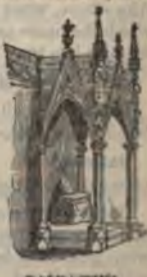
Baldachin. A building in the form of a canopy, supported with columns, and serving as a crown or covering to an altar.

Baluster. A small pillar or column, supporting a rail, of various forms, used in balustrades.

Baluster Shaft. The shaft dividing a window in Saxon architecture. At St. Albans are some of these shafts, evidently out of the old Saxon church, which have been fixed up with Norman capitals.

Balustrade. A series of balusters connected by a rail.

Band. A sort of flat frieze or fascia running horizontally round a tower or other parts of a building, particularly the base tables in perpendicular work, commonly used with the long shafts characteristic of the thirteenth century. It generally has a bold, projecting molding above



BALDACHIN

Barbican. An outwork for the defence of a gate or drawb of pent-house or construction of timber to shelter warders or ser or other missiles.

Barge Board. See *Verge Board*.

Bartizan. A small turret, corbeled out at the angle of a wal tect a warder and enable him to see around him. They generally are furnished with oylets or arrow-slits.

Basement. The lower part of a building, usually in part below the grade of the lot or street.

Base Moldings. The moldings immediately above the plinth of a wall, pillar, or pedestal.

Base of a Column. That part which is between the shaft and the pedestal, or, if there be no pedestal, between the shaft and the plinth. The Grecian Doric had Tuscan has only a single torus, or a plinth.

Basilica. A term given by the Greeks and Romans to the devoted to judicial purposes.

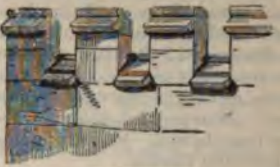
Bas-relief. See *Basso-rilievo*.

Basse-cour. A court separated from the principal one, stables, etc.

Basso-rilievo, or **Bas-relief.** The representations of figur a background without being detached from it. It is divided Alto-rilievo, when the figure projects more than one-half; Mez which the figure projects one-half; and Basso-rilievo, when the figure is less than one-half, as in coins.

Bat. A part of a brick.

Batten. Small scantlings, or small strips of boards, used for



BATTLEMENT

Battlement. A parapet with a series of notches in it, from which arrows may be shot, or other instruments of defence hurled on besiegers. The raised portions are called merlons; and the notches, embrasures or crenelles. The former were intended to cover the soldier while discharging his weapon through the latter. Their use is of great antiquity; they are found in the sculptures of Nineveh, in the tombs of Egypt, and on the famous François vase, where there is a delineation of the siege of Troy. In ecclesiastical architecture the early battlements have small shallow embrasures at some distance apart. In the Decorated period they are closer together, and deeper, and the moldings on the top of the merlon and bottom of the embrasure are richer. During this period, and the early part of the Perpendicular, the sides or cheeks of the embrasures are perfectly square and plain. In later times the moldings were continued round the sides, as well as at top and bottom, mitring at the angles, as over the doorway of Magdalen College, Oxford, England. The battlements of the Decorated and later periods are often richly ornamented by paneling, as in the last example. In castellated work the merlons are often pierced by narrow arrow-slits. (See *Oylet.*) In South Italy some battlements are found strongly resembling those of old Rome and Pompeii; in the Continental ecclesiastical architecture, the parapets are very rarely embattled.

Bay. Any division or compartment of an arcade, roof, etc. Thus each space, from pillar to pillar, in a cathedral, is called a bay, or severy.

Bay Window. Any window projecting outward from the wall of a building, either square or polygonal on plan, and commencing from the ground. If they are carried on projecting corbels, they are called Oriel windows. Their use seems to have been confined to the later periods. In the Tudor and Elizabethan styles they are often semicircular in plan, in which case some think it more correct to call them Bow Windows.

Bazaar. A kind of Eastern mart, of Arabic origin.

Bead. A circular molding. When several are joined, it is called Reeding; when flush with the surface, it is called Quirk-bead; and when raised, Cock-bead.

Beam. A piece of timber, iron, stone, or other material, placed horizontally, or nearly so, to support a load over an opening, or from post to post.

Bearing. The portion of a beam, truss, etc., that rests on the supports.

Bearing Wall, or **Partition.** A wall which supports the floors and roofs in a building.

Beaufet, or **Buffet.** A small cupboard, or cabinet, to contain china. It may either be built into a wall, or be a separate piece of furniture.

Bed. In bricklaying and masonry, the horizontal surfaces on which the stones or bricks of walls lie in courses.

Bed of a Slate. The lower side.

Bed Moldings. Those moldings in all the orders between the corona and frieze.

Belfry. Properly speaking, a detached tower or campanile containing bells, as at Evesham, England, but more generally applied to the ringing-room or loft of the tower of a church. See *Tower.*

Bell-cot, Bell-gable, or **Bell-turret.** The place where one or more be hung in chapels, or small churches which have no towers. Bell-cots are times double, as at Northborough and Coxwell, England; a very commo in France and Switzerland admits of three bells. In these countries, als are frequently of wood, and attached to the ridge. Those which stand gable, dividing the nave from the chancel, are generally called Sanctus Be very curious and, it is believed, unique example at Cleves Abbey, Englan out from the wall. In later times bell-turrets were much ornamented; th often called Flèches.

Bell of a Capital. In Gothic work, immediately above the necking is hollow curve; this is called the bell of a capital. It is often enriched wi ages. It is also applied to the body of the Corinthian and Composite capi

Belt. A course of stones or brick projecting from a brick or stone wa erally placed in a line with the sills of the windows; it is either molded, plane, or enriched with patras at regular intervals. Sometimes called String.

Belvedere, or **Look-out.** A turret or lantern raised above the roo observatory for the purpose of enjoying a fine prospect.

Bema. The semicircular recess, or hexedra, in the basilica, where the sat, and where in after-times the altar was placed. It generally is roofed half-dome or concha. The seats of the priests were against the wall, looki the body of the church, that of the bishop being in the center. The is generally ascended by steps, and railed off by cancelli.

Bench Table. The stone seat which runs round the walls of large ch and sometimes round the piers; it very generally is placed in the porche

Bevel. An instrument for taking angles. One side of a solid body is be beveled with respect to another, when the angle contained between th sides is greater or less than a right angle.

Bezantee. A name given to an ornamental molding much used in th man period, resembling bezants, coins struck in Byzantium.

Billet. A species of ornamented molding much used in Norman, and times in Early English work, like short pieces of stick cut off and arrange nately.

Blocking, or **Blocking-course.** In masonry, a course of stones pla the top of a cornice crowning the walls.

Bond. In bricklaying and masonry, that connection between bricks or formed by lapping them upon one another in carrying up the work, so as t an inseparable mass of building, by preventing the vertical joints fallin each other. In brickwork there are several kinds of bond. In commor walls in every sixth or seventh course the bricks are laid crossways of th called Headers. In face work, the back of the face brick is clipped so as in a diagonal course of headers behind. In Old English bond, every alt course is a header course. In Flemish bond, a header and stretcher alt in each course.

Bond-stones. Stones running through the thickness of the wall at angles to its face, in order to bind it together.

Bond-timbers. Timbers placed in a horizontal direction in the wall brick building in tiers, and to which the battens, laths, etc., are secured. I ble work, walls are better plugged for this purpose.

Border. Useful ornamental pieces around the edge of anything.

Boss. An ornament, generally carved, forming the key-stone at the intersection of the ribs of a groined vault. Early Norman vaults have no bosses. The carving is generally foliage, and resembles that of the period in capitals, etc. Sometimes they have human heads, as at Notre Dame at Paris, and sometimes grotesque figures. In Later Gothic vaulting there are bosses at every intersection.

Boutell. The mediæval term for a round molding, or torus. When it follows a curve, as round a bench end, it is called a Roving Boutell.

Bow. Any projecting part of a building in the form of an arc of a circle. A bow, however, is sometimes polygonal.

Bow Window. A window placed in the bow of a building.

Brace. In carpentry, an inclined piece of timber, used in trussed partitions, or in framed roofs, in order to form a triangle, and thereby stiffen the framing. When a brace is used by way of support to a rafter, it is called a strut. Braces in partitions and span-roofs are, or always should be, disposed in pairs, and introduced in opposite directions.

Brace Mold. [}] Two ressaunts or ogees united together like a brace in printing, sometimes with a small bead between them.

Bracket. A projecting ornament carrying a cornice. Those which support vaulting shafts or cross springers of a roof are more generally called Corbels.

Break. Any projection from the general surface of a building.

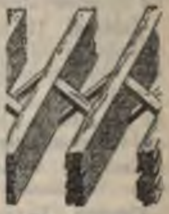
Breaking Joint. The arrangement of stones or bricks so as not to allow two joints to come immediately over each other. See *Bond.*

Breast of a Window. The masonry forming the back of the recess and the parapet under the window-sill.

Bressummer. A lintel, beam, or iron tie, intended to carry an external wall and itself supported by piers or posts; used principally over shop windows. This term is now seldom used, the word *beam*, or *girder*, taking its place.

Bridging. A method of stiffening floor joist and partition studs, by cutting pieces in between. Cross bridging of floor joist is illustrated in cut.

Bulwark. In ancient fortification, nearly the same as Bastion in modern.



CROSS-BRIDGING

Burse, or **Bourse.** A public edifice for the assembly of merchant traders; an exchange.

Bust. In sculpture, that portion of the human figure which comprises the head, neck, and shoulders.

Buttery. A store-room for providions.

Butt-joint. Where the ends of two pieces of timber or molding butt together.

Buttress. Masonry projecting from a wall, and intended to strengthen the same against the thrust of a roof or vault. Buttresses are no doubt derived from the classic pilasters which serve to strengthen walls where there is a pressure of a girder or roof-timber. In very early work they have little projection, and, in fact, are "strippilasters." In Norman work they are wider, with very little projection, and generally stop under a cornice or corbel table. Early English buttresses project considerably, *sometimes with deep sloping weatherings in several*

BUTTRESS

Byzantine Architecture. A style developed in the Byzantine Empire. The capitals of the pillars are of endless variety and full of invention; some are founded on the Greek Corinthian, some resemble the Norman and the Lombard style, and are so varied that no two sides of the same capital are alike. They are comprised under the style Romanesque, which comprehends the round-arch style. Byzantine architecture reached its height in the Church of St. Sophia at Constantinople.

Cabinet. A highly ornamented kind of buffet or chest o for the preservation of things of value.

Cabling. The flutes of columns are said to be cabled w occupied by solid convex masses, or appear to be refilled they had been formed.

Caduceus. Mercury's rod, a wand entwined by two serpe by two wings. The rod represents power; the serpents, and the wings, diligence and activity.

Caisson. A panel sunk below the surface in flat or vaul ings. See *Cassoon*

Caisson. In bridge building, a chest or vessel in which of a bridge are built, gradually sinking as the work advanc bottom comes in contact with the bed of the river, and sides are disengaged, being so constructed as to allow of th thus detached without injury to its floor or bottom.

Caliber, or **Caliper.** The diameter of any round body; t of the mouth of a piece of ordnance.

Camber. In carpentry, the convexity of a beam upon the in order to prevent its becoming concave by its own weigh the burden it may have to sustain.

Campanile. A name given in Italy to the bell-tower of a In that country this is almost always detached from the latt

auze covering over a bed to keep off the gnats; a mosquito curtain.
h canopies are generally simple, with trefoiled or cinque-foiled heads;
ter styles they are very rich, and divided into compartments with
ots, pinnacles, etc. The triangular arrangement over an Early Eng-
corated doorway is often called a canopy. The triangular canopies
of Italy are peculiar. Those in England are generally part of the
of the arch moldings of the door, and form, as it were, the hood-
m, as at York. The former are above and independent of the door
d frequently support an arch with a tympanum, above which is a
anopy, as in the Duomo at Florence. Sometimes the canopy and
from the wall, and are carried on small jamb shafts, as at San Pietro
erona. Canopies are often used over windows, as at York Minster
at west window, and lower ties in the towers. These are triangular,
per windows in the towers have ogee canopies.

The upper part of a column, pilaster, pier, etc. Capitals have been
y style down to the present time. That mostly used by the Egyp-
ll-shaped, with or without ornaments. The Persians used the double-
forming a kind of bracket capital. The Assyrians apparently made
nic and Corinthian, which were developed by the Greeks, Romans,
into their present well-known forms. The Doric was apparently an
r adaptation by the Greeks, and was altered by the Romans and
ut in all these examples, both ancient and modern, the capitals of an
l of the same form throughout the same building, so that if one be
m of all the others is known. The Romanesque architects altered
l in the carving of their capitals often introduced such figures and
helped to tell the story of their building. Another form was intro-
hem in the curtain capital, rude at first, but afterward highly deco-
vidently took its origin from the cutting off of the lower angles of a
, and then rounding them off. The process may be distinctly seen,
l stages, in Mayence Cathedral. But this form of capital was more
ped by the Normans, with whom it became a marked feature. In
nglish capitals a peculiar flower of three or more lobes was used
om the necking upward in most graceful forms. In Decorated and
ar styles this was abandoned in favor of more realistic forms of
aves, enclosing the bell like a wreath. In each style bold abacus
ere always used, whether with or without foliage.

sary. A huge, square building, or inn, in the East, for the reception
and lodging of caravans.

. The timber or iron joist which supports the steps of a wooden stair.

CARYATID

or **Cartoon.** A design made on strong paper, to be
on the fresh plaster wall to be afterward painted in
, a colored design for working in mosaic tapestry.

e. An ornament which like an escutcheon, a shield
or oblong panel has the central part plain, and usually
vex, to receive an inscription, armorial bearings, or an
or significant piece of painting or sculpture. Frequently
ch Renaissance and Modern Architecture.

es. Human female figures used as piers, columns, or
Caryatic is applied to the human figure generally, when
manner of caryatides.

Covered with other materials, generally of a better

Casement. A glass frame which is made to open by turning o affixed to its vertical edges.

Cassoon, or **Caisson.** A deep panel or coffer in a soffit or ceilin term is sometimes written in the French form, *caisson;* sometimes directly from the Italian *cassone,* the augmentative of cassa, a chest o

Cast. A term used in sculpture for the impression of any figure t plaster of Paris, wax, or other substances.

Catacombs. Subterranean places for burying the dead. Those of and near Rome, are believed to be the most important.

Catafalco. An ornamental scaffold used in funeral solemnities.

Cathedral. The principal church, where the bishop has his seat as d

Cauliculus. The inner scroll of the Corinthian capital. It is not unc however, to apply this term to the larger scrolls or volutes also.

Causeway. A raised or paved way.

Cavetto. A concave ornamental molding, opposed in effect to the the quadrant of a circle.

Ceiling. That covering of a room which hides the joists of the floo or the rafters of the roof Most European churches either have open are groined in stone. At Peterborough and St. Albans, England, there old flat ceilings of boards curiously painted. In later times the boarded and, in fact, some of those of plaster, have molded ribs, locked with b the intersection, and are sometimes elaborately carved. In many churches there are ceilings formed of oak ribs, filled in at the s with narrow, thin pieces of board, in exact imitation of stone groin the Elizabethan and subsequent periods the ceilings are enrich most elaborate ornaments in stucco. Matched and beaded boards and smoothed, used for wainscoting. In the New England States it sheathing.

Cenotaph. An honorary tomb or monument, distinguished from mo in being empty, the individual it is to memorialize having received i elsewhere.

Centaur. A poetical imaginary being of heathen mythology, half- half-horse.

Centring. In building, the frames on which an arch is turned.

Chamfer, Champfer, or **Chaumfer.** When the edge or arris of any cut off at an angle of 45° in a small degree, it is said to be cham to a large scale, it is said to be a canted corner. The chamfer is much mediæval work, and is sometimes plain, sometimes hollowed out, and so molded.

Chamfer Stop. Chamfers sometimes simply run into the arris by face; more commonly they are first stopped by some ornament, as by they are sometimes terminated by trefoils, or cinque-foils, double or sin in general form very pleasing features in mediæval architecture.

Chancel. A place separated from the rest of a church by a screen. T is now generally used to signify the portion of an Episcopal or Catholi containing the altar and communion table.

Chantry. A small chapel, generally built out from a church. They contain a founder's tomb, and are often endowed places where for his soul. The officiator, or mass priest, being often unconnected parochial clergy. The chantry has generally an entrance from the

l. A small, detached building used as a substitute for a church in a sh; an apartment in any large building, a palace, a nobleman's house, a r prison, used for public worship; or an attached building running out rming part of a large church, generally dedicated to different saints, ing its own altar, piscina, etc., and screened off from the body of the

r House. The chamber in which the chapter or heads of the monastic sembled to transact business. They are of various forms; some are artments, some octagonal, and some circular.

el. In Gothic architecture, the capital of a pier or column which n arch.

el House. A place for depositing the bones which might be p in digging graves. Sometimes it was a portion of the metimes it was a separate building in the church-yard; s chantry chapels were attached to these buildings. M. Duc has given two very curious examples of *ossuaires* — Fleurance, the other from Faouet.

b — Gothic. A representation of an infant's head joined to s, used in the churches on key-stones of arches and corbels.

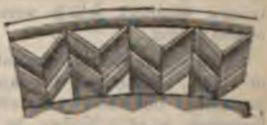
CHAPTREL.

n — Gothic. An ornament turning this and that way, like or letter Z.

-oscuro. The effects of light and a picture.

That part of a church or monastery e breviary service, or "horæ," is

h. A building for the performance of rship. The first churches were built on of the ancient basilicæ, and afterward n of a cross; a church is said to be in Greek cross when the length of verse is equal to that of the nave; in Latin cross, when the nave is n the transverse part; in rotundo, when it is a perfect circle; simple, as only a nave and choir; with aisles, when it has a row of porticos in aulted galleries, with chapels in its circumference.

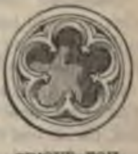


CHEVRON

um. A tabernacle or vaulted canopy supported on shafts standing over altar.

re. A ring, list, or fillet at the top and bottom of a serving to divide the shaft of the column from its d base.

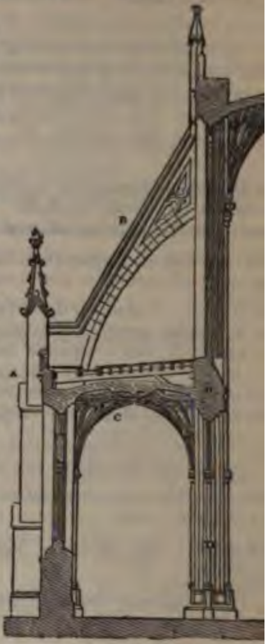
e-foil. A sinking or perforation, like a flower, of s or leaves, as a quatre-foil is of four. The points times in a circle, and sometimes form the cusping



CINQUE-FOIL.

Crown. A garland of oak-leaves and acorns, given ry distinction among the Romans to such as had preserved the life *-citizen.*

Clere-story, Clear-story. When the middle of the nave of a ch above the aisles and is pierced with windows, the upper story is thus called. Sometimes these windows are very small, being mere quatrefoils, or spherical triangles. In large buildings, however, they are important objects both for beauty and utility. The window of the clerestories of Norman work, even in large churches, are of less importance than in the later styles. In Early English they became larger; and in the Decorated they are more important still, being lengthened as the triforium diminishes. In Perpendicular work the latter often disappears altogether, and in many later churches the clerestories are close ranges of windows. The word *clere-story* is also used to denote a similar method of lighting other buildings besides churches, especially factories, depots, sheds, etc.



Bath Abbey

FLYING BUTTRESS AND CLER

A, buttress with pinnacle; buttress supporting clere-s vaulted roof of aisle; D D, pie nave from aisle; E, vaulted roo

Cloister. An enclosed square, like the atrium of a Roman house, with a walk or ambulatory around, sheltered by a roof, generally groined, and by tracery windows, which were more or less glazed.

Close. The precinct of a cathedral or abbey. Sometimes the walls are traceable, but now generally the boundary is only known by tradition.

Close String, or **Box String.** A method of finishing the outer edge of stairs, by building up a sort of curb string on which the balusters set, and the treads and risers stop against it.

Clustered. In architecture, the coalition of several members which penetrate each other.

Clustered Column. Several slender pillars attached to each other so as to form one. The term is used in Roman architecture to denote two or four columns which appear to intersect each other at the angle of a building to answer at each return.

Coat. A thickness or covering of paint, plaster, or other work, done at one time. The first coat of plastering is called the scratch coat, the second coat (when there are three coats) is called the brown coat, and the last coat is variously known as the slipped coat, skimcoat, or white coat. It varies in composition in different localities.

Coffer. A deep panel in a ceiling.

am. A frame used in the building of a bridge in deep water, :aisson.

am. A beam above the lower ends of the rafters, and spiked to

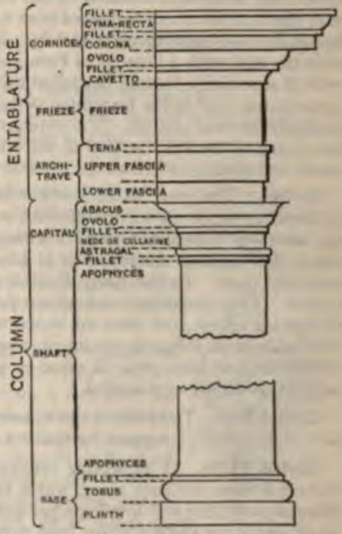
e. A row of columns. The colonnade is termed, according to of columns which support the entablature: Tetrastyle, when there :xastyle, when six; octostyle, when eight, etc. When in front of a / are termed porticos; when surrounding a building, peristyle; and or more, polystyle.

n, or **Coliseum.** The immense amphitheater built at Rome by asian, A D. 72, after his return from his victories over the Jews. It in ninety thousand persons sitting, and twenty thousand more 'he name is now employed unusually large audience nerally of a temporary

The name of a brazen h was erected at the the harbor at Rhodes, and five feet in height. l sail between its legs.

A round pillar. The : base, on which it rests; lled the shaft; and the the capital. The capital a horizontal table, called and the base commonly ıother, called the plinth. y be either insulated or 'hey are said to be at- ngaged when they form ıll, projecting one-half or ıot the whole, of their

A line, angle, surface, elongs equally to several mmon centring is a cen- it trusses, having a tie :om. Common joists are naked flooring to which : fixed. Common rafters those to which the laths are attached.



SECTION OF COLUMN AND ENTABLATURE

(Divided according to the Tuscan Order.)

: Arch. Is the pointed or lancet arch.

: Order. The most elaborate of the orders of classical arch-

A mass composed of broken stone, sand, and hydraulic cement, a sort of artificial stone, much used for foundations; a finer variety used in blocks for building houses.

A long narrow passage between two walls or underground for ınication between different apartments; also, a canal or pipe for the of *water.*

Confessional. The seat where a priest or confessor sits to hear confe

Conge. Another name for the echinus or quarter round.

Conservatory. A building for the protection and rearing of tender
often attached to a house as an apartment. Also, a public place of instr
designed to preserve and perfect the knowledge of some branch of lear
the fine arts; as, a *conservatory* of music.

Consistory. The judicial hall of the College of Cardinals at Rome.

Consol, or **Console.** A bracket or truss, generally with scrolls or vol
the two ends, of unequal size and contrasted, but connected by a flowing line from the back of the upper one to the inner convolving face of the lower.

CONSOLES

Coping. The capping or covering of a wall. This is of stone, weathered to throw off the wet. In Norman times, as far as can be judged from the little there is left, it was generally plain and flat, and projected over the wall with a floating to form a drip. Afterward it assumed a torus or bowtell at the top, and became deeper, and in the Decorated period there were
generally several sets-off. The copings in the Perpendicular period a
something of the wavy section of the buttress caps, and mitred round th
of the embrasure, as well as the top and bottom.

Corbel. The name, in mediæval architecture, for a piece of stone jut
of a wall to carry any superincumbent weight. A piece of timber proje
the same way was called a tassel or a bragger. Thus, the carved ornamen
which the vaulting shafts spring at Lincoln are corbels. Norman cor
generally plain. In the Early English period they are sometimes elab
carved. They sometimes end with a point, apparently growing into t
or forming a knot, and often are supported by angles and other figures.
later periods the foliage or ornaments resemble those in the capitals. In
architecture, a short piece of stone or wood projecting from a wall to
support, generally ornamented.

Corbel Out. To build out one or more courses of brick or stone from
of a wall, to form a support for timbers.

Corbel Table. A projecting cornice or parapet, supported by a r
corbels a short distance apart, which carry a molding, above which is
piece of projecting wall forming a parapet, and covered by a coping. Son
small arches are thrown across from corbel to corbel, to carry the projec

Cornice. The projection at the top of a wall finished by a blocking
common in classic architecture. In Norman times, the wall finished wit
bel table, which carried a portion of plain projecting work, which was
by a coping, and the whole formed a parapet. In Early English times th
pet was much the same, but the work was executed in a much better wa
cially the small arches connecting the corbels. In the Decorated period th
table was nearly abandoned, and a large hollow, with one or two sub
moldings, substituted; this is sometimes filled with the ball-flowers, an
times with running foliages. In the Perpendicular style the parapet fr
did not project beyond the wall-line below; the molding then became
(though often improperly called a cornice), and was ornamented by a qua
or small rosettes, set at equal intervals immediately under the battleme
many French examples the molded string is very bold, and enriched with
ornaments.

The brow of the cornice which projects over the bed moldings to e water.

. A long gallery or passage in a mansion connecting various apart- unning round a quadrangle. Any long passage-way in a building.

ink. To make a cavity for the reception of a plate of iron, or the rew or bolt, so that it shall not project beyond the face of the work.

Columns. Columns arranged in pairs.

A continued layer of bricks or stones in buildings; the term is also o slates, shingles, etc.

An open area behind a house, or in the center of a building and the urts admit of the most elegant ornamentations, such as arcades, etc.

Coving. The molding called the cavetto, or the scotia inverted, on , and not as a mere molding in the composition of a cornice, is called coving.

cketing. The wooden skeleton mold or framing of a cove, applied e bracketing of a cove ceiling.

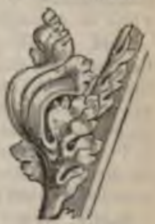
lling. A ceiling springing from the walls with a curve.

nd Flat Ceiling. A ceiling in which the section is the quadrant of ng from the walls and intersecting in a flat surface.

. Timber work for sustaining the lath and plaster of vaulted ceilings.

. An ornamental finish in the wall or ridge of a building, which is the Continent of Europe. An example occurs at Exeter Cathedral, which is ornamented with a range of small fleurs-de-lis in lead.

An ornament running up the sides of gablets, hood-molds, pinna- generally, a winding stem like a creeping plant, s or leaves projecting at intervals, and terminat- al.



CROCKET

This religious symbol is almost always placed on gables, the summit of spires, and other conspicu- f old churches. In early times it was generally often a simple cross in a circle. Sometimes they m of a light cross, crosslet, or a cross in a square. rated and later styles they became richly floriated, d an endless variety of forms. Of memorial finest examples are the Eleanor crosses, erected I. Of these a few yet remain, one of which has n reërected at Charing Cross. Preaching crosses were often set up ide as stations for preaching; the most noted is that in front of St. and. The finest remaining sepulchral crosses are the old elaborately nples found in Ireland.

sle. An old name for a transept.

ringer. The transverse ribs of a vault.

ulting. A common name given to groins and cylindrical vaults.

In architecture the uppermost member of the cornice; called also Larmier.

A vaulted apartment of greater or less size, usually under the choir.

A small room, either circular or polygonal, standing on the top of a some it *is called a Lantern.*

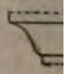
Curb Roof, or **Mansard Roof.** A roof formed of four conti
each two having an external inclination.

Curtail Step. The first step in a stair, which is generally finish
of a scroll.

Cusp. The point where the foliations of tracery intersect.
example in England of a plain cusp is probably that at Pythago
Cambridge, of an ornamental cusp, at Ely Cathedral, where a sm
rosette at the end, is formed at the termination of a cusp. In the l
terminations of the cusps were more richly decorated; they a
terminate not only in leaves or foliages, but in rosettes, heads, and
ornaments.

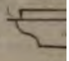
Cyclostyle. A structure composed of a circular range of colu
core is cyclostylar; with a core, the range would be a peristyle.
cies of edifice called by Vitruvius monopteral.

Cyma. The name of a molding of very frequent use. It is a
line, concave at one end and convex at the other, like an
Italic *f*. When the concave part is uppermost it is called
a cyma recta, but if the convexity appear above, and the
concavity below, it is then a cyma reversa.



CY

Cymatium. When the crowning molding of an entablature is of the cyma form, it is termed the Cymatium.



CYM

Cyrtostyle. A circular projecting portico. Such are those of the transept entrances to St. Paul's Cathedral, London.

Dado, or **Die.** The vertical face of an insulated pedestal be
and cornice, or surbase. It is extended also to the similar part of
which are arranged like pedestals in Roman and Italian architec

Dais. A part of the floor at the end of a mediæval hall, rais
the rest of the floor. On this the lord of the mansion dined wit
the great table, apart from the retainers and servants. In medi
was generally a deep recessed bay window at one or at each e
supposed to be for retirement, or greater privacy than the open h
In France the word is understood as a canopy or hanging over a
the name was given from the fact that the seats of great men
mounted by such an ornament.

Darby. A flat tool used by plasterers in working, especially
is generally about seven inches wide and forty-two inches long, w
on the back.

Decastyle. A portico of ten columns in front.

Decorated Style. The second stage of the Pointed or Gothi
tecture, considered the most complete and perfect development o
tecture, the best examples of which are found in England.

Demi-metope. The half of a metope, which is found at the
jecting angles of a Doric frieze.

il. As used by architects, detail means the smaller parts into which a ition may be divided. It is applied generally to moldings and other ents, and again to their minutiæ.

ieter. The line in a circle passing through its center, or thickest part, ives the measure proportioning the intercolumniation in some of the

ieters. The diameters of the lower and upper ends of the shaft of a are called its inferior and superior diameters, respectively; the former is test, the latter the least diameter of the shaft.

er. A method of decorating a wall, panel, stained glass, or any plain sur- covering it with a continuous design of flowers, rosettes, etc., either in or lozenges, or some geometrical form resembling the pattern of a dia- ble-cloth, from which, in fact, the name is supposed by some to have rived.

yle. A spacious intercolumniation, to which three diameters are l.

ros. A double-winged temple. The Greeks are said to have constructed with two ranges of columns all around, which were called dipteroi. A projecting two columns and their interspaces is of dipteral or pseudo- arrangement.

iarging Arch. An arch over the opening of a door or window, to dis- r relieve the superincumbent weight from pressing on the lintel.

mper. Term applied to painting with colors mixed with size or other is substance. All the cartoons of the ancients, previous to the year 1410, to be done in distemper.

le. A portico of two columns. This is not generally applied to the mere ith two columns, but to describe a portico with two columns *in antis*.

glyph. An intercolumniation in the Doric order, of two triglyphs.

castyle. A portico of twelve columns in front. The lower one of the nt of St. Paul's Cathedral, London, is of twelve columns, but they are , making the arrangement pseudo-dodecastyle. The Chamber of Depu- Paris has a true dodecastyle.

tooth. A favorite enrichment used from the latter part of the Norman o the early part of the Decorated. It is in the form of a four-leaved the center of which projects, and probably was named from its resem- o the dog-toothed violet.

e. A cupola or inverted cup on a building. The application of this term nerally received purpose is from the Italian custom of calling an archi- l church, by way of eminence, Il Duomo, the temple; for to one of that e Cathedral of Florence, the cupola was first applied in modern practice. lians themselves never call a cupola a dome; it is on this side of the Alps lication has arisen, from the circumstance, it would appear, that the Ital- the term with reference to those structures whose most distinguishing is the cupola, tholus, or (as we now call it) dome.

estic Architecture. That branch which relates to private buildings.

on. The principal tower of a castle, generally containing the prison.

Frame. The surrounding case into and out of which the door shuts and It consists of two upright pieces, called jambs, and a head, generally fixed by mortices and tenons, and wrought, rebated, and beaded.

Order. *The oldest of the three orders of Grecian architecture.*

some of the Elizabethan, the entire upper story in the roof apartment, said to have been a place for exercise in wet weat dormitory for the retainers of the household, or those of visit

Double Vault. Formed by a duplicate wall; wine cellars formed.

Dovetailing. In carpentry and joinery, the method of fa other timbers together, by letting one piece into another ir expanded tail of a dove.

Dowel. A pin let into two pieces of wood or stone, whe together. A piece of wood driven into a wall so that other pi to it. This is also called plugging.

Draw-bridge. A bridge made to draw up or let down, mu places. In navigable rivers, the arch over the deepest chann or revolve, in order to let the masts of ships pass through.

Drawing-room. A room appropriated for the reception of to which company withdraws from the dining-room.

Dresser. A cupboard or set of shelves to receive dishes an

Dressing. Is the operation of squaring and smoothing st also applied to smoothing lumber.

Dressing-room. An apartment appropriated for dressing

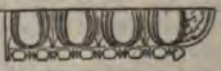
Drip. A name given to the member of a cornice which beyond the other parts for throwing off water by small porti It is also called Larmier.

Drip-stone. The label molding which serves on a canop and to throw off the rain. It is also called Weather Molding

Drop-scene. A curtain suspended by pulleys, which des front of the stage in a theater.

Drum. The upright part of a cupola over a dome; also, the of the Corinthian and Composite capitals.

In slating and shingling, the margin or lower part of the slating ›ver the wall, to throw the water off from the masonry or brickwork.



ECHINUS

ıs. A molding of eccentric curve, gener- when it is carved) into the forms of eggs ıors alternating, whence the molding is the name of the more conspicuous. It is as Ovolo.

:. Is synonymous with the terms building, fabric, erection, but is ictly applicable to architecture distinguished for size, dignity, and

ıscence. In architecture, the formation of a whitish loose powder, or the surface of stone or brick walls.

an Architecture. The earliest civilization and cultivation of the arts ɔper Egypt. The most remarkable and most ancient monuments of the s, with the exception of the pyramids, are nearly all included in Upper The buildings of Egypt are characterized by solidity and massiveness ıction, originality of conception, and boldness of form. The walls, the ıd the most sacred places of their religious buildings were ornamented ·oglyphics and symbolical figures, while the ceilings of the porticos zodiacs and celestial planispheres. The temples of Egypt were gener- ɔut roofs, and, consequently, the interior colonnades had no pediments, ıg merely an entablature, composed of only architrave, frieze, and ormed of immense blocks united without cement and ornamented with hics.

nt. The outline of the design of a Decorated window, on which the ɔr the tracery are formed. These centers will all be found to fall on ıich, in some way or other, will be equimultiples of parts of the openings. tracery well, or understand even the principles of its composition, much should be given to the study of the element.

ion. The front façade, as the French term it, of a structure; a geo- drawing of the external upright parts of a building.

ttlement. An indented parapet; battlement.

zon. To adorn with figures of heraldry, or ensigns armorial.

ssing. Sculpture in rilievo, the figures standing partly out from the

ısure. The opening in a battlement between the two raised solid por- nerlons, sometimes called a crenelle.

stic. Pertaining to the art of burning in colors, applied to painting on ·celain, or tiles, where colors are fixed by heat; hence, encaustic tiles, c.

ed Columns. Are those attached to, or built into walls or piers, a eing concealed.

ıment. The addition of ornament, carving, etc., to plain work; decora- ɔellishment.

ıble. Means the whole work or composition considered together, and rts.

ature. The assemblage of parts supported by the column. It con- ıree parts: the architrave, frieze, and cornice.

In Gothic architecture, delicate carving.

Entasis. The swelling of a colu
spires, particularly those called "br
sides, but no more than to make t
"deceptio visus," that which is qui
hollow.

Entry. A hall without stairs or

Epistyle. This term may with pr
with which it is synonymous; but
lowest member of the entablature.

Escutcheon. (Her.) The field o
sented. (Arch.) The shields used
string-courses; also, the ornamentec
knockers, etc., are suspended, or wh
the wear of the key. In mediæval
beautiful manner.

Etching. A mode of engraving o
of lines, eaten in or corroded by me

Eustyle. A species of intercolum
eters and a quarter is assigned. T
import — pycnostyle, systyle, diasty
of columns from one another in co
each the space it is to express. It
individually, that the words them
able convey no idea of an exactl
columnar structures of the ancient
such limitations. It follows, then,
purely conventional, and may or m
power of applying the terms. Eu
rangement; but, as the effect of
things besides the diameter of th
columniation would look well or i
so that the limitation of Eustyle

Extrados. The exterior or conv
stones; the term is opposed to the i

Eye of a Dome. The aperture

Eye of a Volute. The circle in

Façade, or **Face.** The whole ex
one view; strictly speaking, the pri

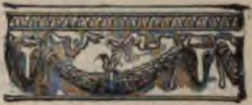
Face Mold. The pattern for
ornamental hand-railings for stairs

Fan Tracery. The very compli
dicular style, in which the vault is

Fascia. A flat, broad member in
buildings, but of small projection.
composed of three bands, or fasciæ;
one. Ornamental projections from
windows, except the uppermost, are

Fenestral. A frame, or "chas
strained to keep out wind and

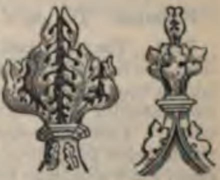
Festoon. An ornament of carved work, representing a wreath or garland of flowers or leaves, or both, interwoven with each other. It is thickest in the middle, and small at each extremity, where it is tied, a part often hanging down below the knot.



FESTOON

Fillet. A narrow vertical band or listel of frequent use in congeries of moldings, to separate and combine them, and also to give breadth and firmness to the upper edge of a crowning cyma or cavetto, as in an external cornice. The narrow slips or breadth between the flutes of Corinthian and Ionic columns are also called fillets. In mediæval work the fillet is a small, flat, projecting square, chiefly used to separate hollows and rounds, and often found in the outer parts of shafts and boutels. In this situation the center fillet has been termed a keel, and the two side ones, wings; but, apparently, this is not an ancient usage.

Finial. The flower, or bunch of flowers, with which a spire, pinnacle, gablet, canopy, etc., generally terminates. Where there are crockets, the finial generally bears as close a resemblance as possible to them in point of design. They are found in early work where there are no crockets. The simplest form more resembles a bud about to burst than an open flower. They soon became more elaborate, as at Lincoln, and still more, as at Westminster and the Hôtel Cluny at Paris. Many perpendicular finials are like four crockets bound together. Almost every known example of a finial has a sort of necking separating it from the parts below.



FINIALS

Fish-joint. A splice where the pieces are joined butt end to end, and are connected by pieces of wood or iron placed on each side and firmly bolted to the timbers, or pieces joined.

Flags. Flat stones, from 1 to 3 inches thick, for floors.

Flamboyant. A name applied to the Third Pointed style in France, which seems to have been developed from the Second, as the English Perpendicular was from the Decorated. The great characteristic is, that the element of the tracery flows upward in long wavy divisions like flames of fire. In most cases, also, every division has only one cusp on each side, however long the division may be. The moldings seem to be as much inferior to those of the preceding period as the Perpendicular moldings were to the Early English, a fact which seems to show that the decadence of Gothic architecture was not confined to one country.

Flange. A projecting edge, rib, or rim. Flanges are often cast on the top or bottom of iron columns, to fasten them to those above or below; the top and bottom of I-beams and channels are called the flange.

Flashings. Pieces of lead, tin, or copper, let into the joints of a wall so as to lap over gutters or other pieces; also, pieces worked in the slates or shingles around dormers, chimneys, and any rising part, to prevent leaking.

Flatting. Painting finished without leaving a gloss on the surface.

Flèche. A general term in French architecture for a spire, but more particularly used for the small, slender erection rising from the intersection of the nave and transepts in cathedrals and large churches, and carrying the sanctus bell.

Fleur-de-lis. *The royal insignia of France, much used in decoration.*

Flight. A run of steps or stairs from one landing to another.

Floating. The equal spreading of plaster or stucco on the surface of by means of a board called a float; as a rule, only rough plastering is flo

Floriated. Having florid ornaments, as in Gothic pillars.

Flue. The space or passage in a chimney through which the smoke as Each passage is called a flue, while all together make the chimney.

Flush. The continued surface, in the same plane, of two contiguous

Flute. A concave channel. Columns whose shafts are channeled a to be fluted, and the flutes are collectively called Flutings.

Flying Buttress. An arched buttress used when extra strength was r for the upper part of the wall of the nave, etc., to resist the outward thru vaulted ceiling. The flying buttress generally rests on the wall and butt the aisle.

Foils. The small arcs in the tracery of Gothic windows, panels, etc.

Foliage. An ornamental distribution of leaves on various parts of bui

Foliation. The use of small arcs or foils in forming tracery.

Font. The vessel used in the rite of baptism. The earliest extant is su to be that in which Constantine is said to have been baptized; this is a po labrum from a Roman bath. Those in the baptisteries in Italy are all lar were intended for immersion; as time went on, they seem to have smaller. Fonts are sometimes mere plain hollow cylinders, generally smaller below than above; others are massive squares, supported on a thic round which sometimes there are smaller shafts. In the Early English th is still pursued, and the shafts are detached; sometimes, however, they a agonal and octagonal, and in this and the later styles assume the form of on a stem. Norman fonts frequently have curious carvings on them, app ing the grotesque; in later times the foliages, etc., partook absolutely character of those used in other architectural details of their respective The font in European churches is usually placed close to a pillar near t trance, generally that nearest but one to the tower in the south arcade; large buildings, in the middle of the nave, opposite the entrance porc sometimes in a separate building. In Protestant churches in this count font is generally placed inside the communion rail, or on the steps of the c

Footings. The spreading courses at the base or foundation of a wall. a layer of different material from that of the wall (as a bed of concrete) i it is called the Footing.

Foundation. That part of a building or wall which is below the sur the ground.

Foxtail Wedging. Is a peculiar mode of mortising, in which the end tenon is notched beyond the mortise, and is split and a wedge inserted, being forcibly driven in, enlarges the tenon and renders the joint firm a movable.

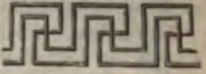
Frame. The name given to the wood-work of windows, doors, etc.; carpentry, to the timber works supporting floors, roofs, etc.

Framing. The rough timber work of a house, including the flooring, r partitioning, ceiling, and beams thereof.

Freestone. Stone which can be used for moldings, tracery, and othe required to be executed with the chisel. The oölitic and sandstones are generally included by this term.

Fresco. The method of painting on a wall while the plastering is wet. The color penetrates through the material, which, therefore, will bear rubbing or cleaning to almost any extent. The transparency, the chiaro-oscuro, and lucidity, as well as force, which can be obtained by this method, cannot be conceived unless the frescos of Fra Angelico or Raphael are studied. The word, however, is often applied improperly to painting on the surface in distemper or body color, mixed with size or white of egg, which gives an opaque effect.

Fret. An ornament consisting of small fillets intersecting each other at right angles.



FRET

Frieze. That portion of an entablature between the cornice above and architrave below. It derives its name from being the recipient of the sculptured enrichments either of foliage or figures which may be relevant to the object of the sculpture. The frieze is also called the Zoöphorus.

Frigidarium. An apartment in the Roman bath, supplied with cold water.

Furniture. A name given to the metal trimmings of doors, windows, and other similar parts of a house. In this country the word "hardware" is more generally used to denote the same thing.

Furrings. Flat pieces of timber used to bring an irregular framing to an even surface.

Gable. When a roof is not hipped or returned on itself at the ends, its ends are stopped by carrying up the walls under them in the triangular form of the roof itself. This is called the gable, or, in the case of the ornamental and ornamented gable, the pediment. Of necessity, gables follow the angles of the slope of the roof, and differ in the various styles. In Norman work they are generally about half-pitch; in Early English, seldom less than equilateral, and often more. In Decorated work they become lower, and still more so in the Perpendicular style. In all important buildings they are finished with copings or parapets. In the Later Gothic styles gables are often surmounted with battlements, or enriched with crockets; they are also often paneled or perforated, sometimes very richly. The gables in ecclesiastical buildings are mostly terminated with a cross; in others, by a finial or pinnacle. In later times the parapets or copings were broken into a sort of steps, called corbic steps. In buildings of less pretension the tiles or other roof covering passed over the front of the wall, which then, of course, had no coping. In this case, the outer pair of rafters were concealed by molded or carved verge boards.

Gable Window. A term sometimes applied to the large window under a gable, but more properly to the windows in the gable itself.

Gabled Towers. Those which are finished with gables instead of parapets. Many of the German Romanesque towers are gabled.

Gablets. Triangular terminations to buttresses, much in use in the Early English and Decorated periods, after which the buttresses generally terminate in pinnacles. The Early English gablets are generally plain, and very sharp in pitch. In the Decorated period they are often enriched with paneling and crockets. They are sometimes finished with small crosses, but oftener with finials.

Gain. A beveled shoulder on the end of a mortised brace, for the purpose of giving additional resistance to the shoulder.

Gallery. Any long passage looking down into another part of a building, or into the court outside. In like manner, any stage erected to carry a rood or an organ, or to receive spectators, was latterly called a gallery, though originally a

loft. In later times the name was g
those intended for purposes of state,

Gambrel Roof. A roof with two

Gargoyle, or **Gurgoyle.** The ca
to a spout which conveyed away th
gutters, supposed to be called so fr
noise made by the water passing t
goyles are mostly grotesque figures.

Gate-house. A building formin
a town, the door of an abbey, or t
castle or other important edifice. T
a large gateway protected by a gate
cullis, over which were battlement
holes (machicolations) for throwing d
lead, or hot sand on the besiege
always had a lodge, with apartmen
and guard-rooms for the soldiers;
rooms over for the officers, and ofte
now commonly applied to the gate-

Gauge. To mix plaster of Paris
called gauged mortar. A tool used
edge of a board.

Girder. A large timber or iron
port joists or walls over an opening.

Glyph. A vertical channel in a f

Gothic Style. The name of Got
at a period in the sixteenth century
and everything not classic was cons
thus originally intended as one of sti
the character of the Mediæval style
from the Grecian and Roman. The
vertical division, relation and subor
yet at unity with each other, and
architecture may be said to have re

Grange. A word derived from th
Granges were usually long buildings
by posts or columns into a sort of na
In England the term was applied n
buildings which formed the detach
most cases there was a chapel either
separate edifice.

Grillage. A framework of beam
beams notched upon them, used to

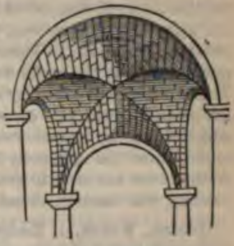
Grille. The iron-work forming t
tecting railing to a tomb or shrine;
England. They are of wrought iron
put together either by rivets or clip
sively for protecting the lower wind
outside doors.

Groin. By some described as t
cross each other, which others cal

or spandrel of such vaulting is called a groin, and by others the whole system of vaulting is so named.

Groin Arch. The cross-rib in the later styles of groining, passing at right angles from wall to wall, and dividing the vault into bays or travees.

Groin Ceiling. A ceiling to a building composed of oak ribs, the spandrels of which are filled in with narrow, thin slips of wood. There are several in England; one at the Early English church at Warmington, and one at Winchester Cathedral, exactly resembling those of stone.



GROINED VAULTING

Groin Centring. In groining without ribs, the whole surface is supported by centring during the erection of the vaulting. In ribbed work the stone ribs only are supported by timber ribs during the progress of the work, any light stuff being used while filling in the spandrels.

Groin Point. The name given by workmen to the arris or line of intersection of one vault with another where there are no ribs.

Groin Rib. The rib which conceals the groin point or joints, where the spandrels intersect.

Groined Vaulting. The system of covering a building with stone vaults which cross and intersect each other, as opposed to the barrel vaulting, or series of arches placed side by side. The earliest groins are plain, without any ribs, except occasionally a sort of wide band from wall to wall, to strengthen the construction. In later Norman times ribs were added on the line of intersection of the spandrels, crossing each other, and having a boss as a key common to both; these ribs the French authors call *nerfs en ogive*. Their introduction, however, caused an entire change in the system of vaulting; instead of arches of uniform thickness and great weight, these ribs were first put up as the main construction, and spandrels of the lightest and thinnest possible material placed upon them, the haunches only being loaded sufficiently to counterbalance the pressure from the crown. Shortly after, half-ribs against the walls (formerets) were introduced to carry the spandrels without cutting into the walling, and to add to the appearance. The work was now not treated as continued vaulting, but as divided into bays, and it was formed by keeping up the ogive, or intersecting ribs and their bosses; a sort of construction having some affinity to the dome was formed, which added much to the strength of the groining. Of course, the top of the soffit or ridge of the vault was not horizontal, but rose from the level of the top of the formeret-rib to the boss and fell again; but this could not be perceived from below. As this system of construction got more into use, and as the vaults were required to be of greater span and of higher pitch, the spandrels became larger, and required more support. To give this, another set of ribs was introduced, passing from the springers of the ogive ribs, and going to about half-way between these and the ogive, and meeting on the ridge of the vault; these intermediate ribs are called by the French *tiercerons*, and began to come into use in the transition from Early English to Decorated. About the same period a system of vaulting came into use called *hexpartite*, from the fact that every bay is divided into six compartments instead of four. It was invented to cover the naves of churches of unusual width. The filling of the spandrels in this style is very peculiar, and, where the different compartments meet at the ridge, some pieces of harder stone have been used, *which give rather a pleasing effect.* The arches against the wall, *being of smaller span than the main arches,* cause the centre springers to be per-

pendicular and parallel for some height, and the spandrels themselves are
hollow. As styles progressed, and the desire for greater richness increa
another series of ribs, called *liernes*, was introduced; these passed crossways f
the *ogives* to the *tiercerons*, and thence to the *doubleaux*, dividing the span
nearly horizontally. These various systems increased in the Perpendic
period, so that the walls were quite a net-work of ribs, and led at last to
Tudor, or, as it is called by many, fan-tracery vaulting. In this system the
are no part of the real construction, but are merely carved upon the vouss
which form the actual vaulting. Fan Tracery is so called because the
radiate from the springers, and spread out like the sticks of a fan. These
methods are not strictly groins, for the pendentives are not square on plan,
circular, and there is, therefore, no arris intersection or groin point.

Groins, Welsh, or **Underpitch.** When the main longitudinal vault of
groining is higher than the cross or transverse vaults which run from the wind
the system of vaulting is called underpitch groining, or, as termed by the w
men, Welsh groining. A very fine example is at St. George's Chapel, Win
England.

Groove. In joinery, a term used to signify a sunk channel whose secti
rectangular. It is usually employed on the edge of a molding, stile, or
etc., into which a tongue corresponding to its section, and in the substan
the wood to which it is joined, is inserted.

Grotesque. A singular and fantastic style of ornament found in an
buildings.

Grotto. An artificial cavern.

Ground Floor. The floor of a building on a level, or nearly so, with
ground.

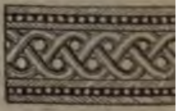
Ground Joist. Joist that is blocked up from the ground.

Grounds. Pieces of wood embedded in the plastering of walls to w
skirting and other joiner's work is attached. They are also used to stop
plastering around door and window openings.

Grouped Columns. Three, four, or more columns put together on the
pedestal. When two are placed together, they are said to be coupled.

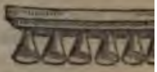
Grout. Mortar made so thin by the addition of water that it will run into all the joints and cavities of the mason-work, and fill it up solid.

Guilloche, or Guillochos. An interlaced ornament like net-work, used most frequently to enrich the torus.



GUILLOCHE

Guttæ. The small cylindrical drops used to enrich the mutules and regulæ of the Doric entablature are so called.



GUTTÆ

Gutter. The channel for carrying off rain-water. The mediæval gutters differed little from others, except that they are often hollows sunk in the top of stone cornices, in which case they are generally called channels in English, and *cheneaux* in French.

Gymnasium. A building classed in the first rank by the Greeks; it w
them they instructed the youth in all the arts of peace and war; a buildin
athletic exercises.

The principal apartment in the large dwellings of the Middle Ages, the purposes of receptions, feasts, etc. In the Norman castle the hall rally in the keep above the ground floor, where the retainers lived, the t being devoted to stores and dungeons for confining prisoners. Later ndeed, some Norman halls (not in castles) — are generally on the ground at Westminster, approached by a porch either at the end, as in this last or at the side, as at Guildhall, London, having at one end a raised dais le. The roofs are generally open and more or less ornamented. In the f these was an opening to let out the smoke, though in later times the e large chimney-places with funnels or chimney-shafts for this purpose. period there were usually two deeply recessed bay windows at each end is, and doors leading into the withdrawing-rooms, or the ladies' apart-hey are also generally wainscoted with oak, in small panels, to the height r six feet, the panels often being enriched. Westminster Hall was y divided into three parts, like a nave and side aisles, as are some on the it of Europe. A room or passage-way at the entrance of a house, or chambers. A place of public assembly, as a town-hall, a music-hall.

ng. The junction of two pieces of timber, by letting one into the other.

ner Beam. A beam in a Gothic roof, not extending to the opposite beam at the foot of a rafter.

ing Buttress. A buttress not rising from the ground, but supported on applied chiefly as a decoration and used only in the Decorated and icular style.

ing Stile. Of a door, is that to which the hinges are fixed.

ings. Tapestry; originally invented to hide the coarseness of the walls mber. Different materials were employed for this purpose, some of ceedingly costly and beautifully worked in figures, gold and silk.

ing. Drawing parallel lines close together for the purpose of indicat-tion of anything. The lines are generally drawn at an angle of 45° with ntal.

ches. The sides of an arch, about half-way from the springing to the

ers. In masonry, are stones or bricks extending over kness of a wall. In carpentry, the large beam into e common joists are framed in framing openings for himneys, etc.

ing Courses. Courses of a wall in which the stone are all headers.

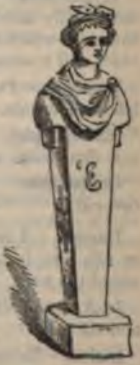
-way. Clear space or height under an arch, or over ay, and the like.

Of a rafter, the end or foot that rests upon the te.

ht. Of an arch, a line drawn from the middle of the the intrados.

A small volute or twist like a stalk, representing ted tops of the acanthus, placed under the abacus of nthian capital.

es. A rough quadrangular stone or pillar, having usually of Hermes or Mercury, sculptured on the *hout arms or body, placed by the Greeks in front of*



HERMES

Herring-bone Work. Bricks, tile, or other materials arranged diagon in building.

Hexastyle. A portico of six columns in front is of this description.

High Altar. The principal altar in a cathedral or church. Where there second, it is generally at the end of the choir or chancel, not in the lady ch

Hip-knob. The finial on the hip of a roof, or between the barge boards gable.

Hip-roof. A roof which rises by equally inclined planes from all four of the building.

Hippodrome. A place appropriated by the ancients for equestrian exer

Hips. Those pieces of timber placed in an inclined position at the corne angles of a hip-roof.

Hood-mold. A word used to signify the drip-stone for label over a wi or door opening, whether inside or out.

Hôtel de Ville. The town-hall, or guild-hall, in France, Germany, Northern Italy. The building, in general, serves for the administration of jus the receipt of town dues, the regulation of markets, the residence of magistr barracks for police, prisons, and all other fiscal purposes. As may be imagi they differ very much in different towns, but they have almost invari attached to them, or closely adjacent, a large clock-tower containing on more bells, for calling the people together on special occasions.

Hôtel Dieu. The name for a hospital in mediæval times. In England t are but few remains of these buildings, one of which is at Dover; in France t are many. The most celebrated is the one at Angers, described by Pa They do not seem to differ much in arrangement of plan from those in mo days, the accommodation for the chaplain, medicine, nurses, stores, etc., much the same in all ages, except that in some of the earlier, instead of the being placed in long wards like galleries, as is now done, they occupied buildings, with naves and side aisles, like churches.

Housing. The space taken out of one solid to admit the insertion of ano The base on a stair is generally housed into the treads and risers; a niche statue.

Hypæthros. A temple open to the air, or uncovered. The term may b more easily understood by supposing the roof removed from over the nave church in which columns or piers go up from the floor to the ceiling, leavin aisles still covered.

Hypogea. Constructions under the surface of the earth, or in the sides hill or mountain.

Ichnography. A horizontal section of a building or other object, showi true dimensions according to a geometric scale, a ground plan.

Impluvium. The central part of an ancient Roman court, which wa covered.

Impost. A term in classic architecture for the horizontal moldings of or pilasters, from the top of which spring the archivolts or moldings whi round the arch.

In Antis. When there are two columns between the antæ of the lateral and the cella.

Incise. To cut in; to carve; to engrave.

Indented. Toothed together.

Inlaying. Inserting pieces of ivory, metal, or choice woods, or the like, into a groundwork of some other material, for ornamentation.

Insulated. Detached from another building. A church is insulated, when not contiguous to any other edifice. A column is said to be insulated, when standing free from the wall; thus, the columns of peripteral temples were insulated.

Intaglio. A sculpture or carving in which the figures are sunk below the general surface, such as a seal the impression of which in wax is in bas-relief; opposed to Cameo.

Intercolumniation. The distance from column to column, the clear space between columns.

Interlaced Arches. Arches where one passes over two openings, and they consequently cut or intersect each other.

Intrados. Of an arch, the inner or concave curve of the arch stones.

Inverted Arches. Those whose key-stone or brick is the lowest in the arch.

Ionic Order. One of the orders of Classical architecture.

Iron Work. In mediæval architecture, as an ornament, is chiefly confined to the hinges, etc., of doors and of church chests, etc. In some instances not only do the hinges become a mass of scroll work, but the surface of the doors is covered by similar ornaments. In almost all styles the smaller and less important doors had merely plain strap hinges, terminating in a few bent scrolls, and latterly in fleurs-de-lis. Escutcheon and ring handles, and the other furniture, partook more or less of the character of the time. On the Continent of Europe the knockers are very elaborate. At all periods doors have been ornamented with nails having projecting heads, sometimes square, sometimes polygonal, and sometimes ornamented with roses, etc. The iron work of windows is generally plain, and the ornament confined to simple fleur-de-lis heads to the stanchions. The iron work of screens enclosing tombs and chapels is noticed under *Grille, q.v.*

Jack. An instrument for raising heavy loads, either by a crank, siren and pinion, or by hydraulic power, and in all cases worked by hand.

Jack Rafter. A short rafter, used especially in hip-roofs.

Jamb. The side-post or lining of a doorway or other aperture. The jambs of a window outside the frame are called Reveals.

Jamb-shafts. Small shafts to doors and windows with caps and bases; when in the inside arris of the jamb of a window they are sometimes called Esconsons.

Joggle. A joint between two bodies so constructed by means of jogs or notches as to prevent their sliding past each other.

Joinery. That branch in building confined to the nicer and more ornamental parts of carpentry.

Joist. A small timber to which the boards of a floor or the laths of ceiling are nailed. It rests on the wall or on girders.

Keep. The inmost and strongest part of a mediæval castle, answering to the citadel of modern times. The arrangement is said to have originated with Gundulf, the celebrated Bishop of Rochester. The Norman keep is generally a very massive square tower, the basement or stories partly below ground being used for stores and prisons. The main story is generally a great deal above ground level, with a projecting entrance, approached by a flight of steps and drawbridge. This floor is generally supposed to have been the guard-room or place for the soldiery; above this was the hall, which generally extended over the whole area of the building, and is sometimes separated by columns; above this are other apartments for the residents. There are winding staircases in the angles of the

buildings, and passages and small chambers in the thickness of the walls.
keep was intended for the last refuge, in case the outworks were scaled a
other buildings stormed. There is generally a well in a mediæval keep,
iously concealed in the thickness of a wall, or in a pillar. The most cele
of Norman times are the White Tower in London, the castles at Roc
Arundel, and Newcastle, Castle Hedingham, etc. The keep was often ci

Key-stone. The stone placed in the center of the top of an arch. Th
acter of the key-stone varies in different orders. In the Tuscan and Dor
only a simple stone projecting beyond the rest; in the Ionic it is adorne
moldings in the manner of a console; in the Corinthian and Composite
rich-sculptured console.

King-post. The middle post of a trussed piece of framing for supporti
tie-beam at the middle and the lower ends of the struts.

Knee. A piece of timber naturally or artificially bent to receive anot
relieve a weight or strain.

Knob, Knot. The bunch of flowers carved on a corbel, or on a Boss.

Kremlin. The Russian name for the citadel of a town or city.

Label. Gothic; the drip or hood-molding of an arch, when it is retur
the square.

Label Terminations. Carvings on which the labels terminate ne
springing of the windows. In Norman times those were frequently gro
heads of fish, birds, etc., and sometimes stiff foliage. In the Early Engli
Decorated periods they are often elegant knots of flowers, or heads of
queens, bishops, and other persons supposed to be the founders of churche
the Perpendicular period they are often finished with a short square, mitred
or knee, and the foliages are generally leaves of square or octagonal form

Lacunar. A paneled or coffered ceiling or soffit. The panels or cass
a ceiling are by Vitruvius called lacunaria.

LACUNARS IN CE

Lady-chapel. A small chapel dedicated to the Virgin Mary, generally found in ancient cathedrals.

Lancet. A high and narrow window pointed like a lancet, often called a lancet window.

Landing. A platform in a flight of stairs between two stories; the terminating of a stair.

Lantern. A turret raised above a roof or tower and very much pierced, the better to transmit light. In modern practice this term is generally applied to any raised part in a roof or ceiling containing vertical
windows, but covered in horizontally. The name was also often applied
louver or femerell on a roof to carry off the smoke; sometimes, too, to th
constructions at the top of towers, as at Ely Cathedral, probably becaus
were placed in them at night to serve as beacons.

Lanterns of the Dead. Curious small slender towers, found chiefly
centre and west of France, having apertures at the top, where a light
hibited at night to mark the place of a cemetery. Some have supposed t
round towers in Ireland may have served for this purpose.

Lath. A slip of wood used in slating, tiling, and plastering.

Lattice. Any work of wood or metal made by crossing laths, rods,
and forming a net-work. A reticulated window, made of laths or sli

separated by glass windows, and only used where air rather than light is to be admitted, as in cellars and dairies.

Lavabo. The lavatory for washing hands, generally erected in cloisters of monasteries. A very curious one at Fontenay, surrounding a pillar, is given by Viollet-le-Duc. In general, it is a sort of trough, and in some places has an almry for towels, etc.

Lavatory. A place for washing the person.

Lean-to. A small building whose rafters pitch or lean against another building, or against a wall.

Lectern. The reading-desk in the choir of churches.

Ledge, or **Ledgement.** A projection from a plane, as slips on the side of window and door frames to keep them steady in their places.

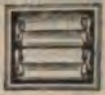
Ledgers. The horizontal pieces fastened to the standard poles or timbers of scaffolding raised around buildings during their erection. Those which rest on the ledgers are called putlogs, and on these the boards are laid.

Lewis. An iron clamp dovetailed into a large stone to lift it by.

Lich-gate. A covered gate at the entrance of a cemetery, under the shelter of which the mourners rested with the corpse, while the procession of the clergy came to meet them. There are several examples in England.

Light. A division or space in a sash for a single pane of glass; also a pane of glass.

Linen Scroll. An ornament formerly used for filling panels, and so called from its resemblance to the convolutions of a folded napkin.


LINEN SCROLL

Lining. Covering for the interior, as casing is covering for the exterior surface of a building; also, such as linings of a door for windows, shutters, and similar work.

Lintel. The horizontal piece which covers the opening of a door or window.

Lip Mold. A molding of the Perpendicular period like a hanging lip.

List, or **Listel.** A little square molding, to crown a larger; also termed a fillet.

Lithograph. A print from a drawing on stone.

Lobby. An open space surrounding a range of chambers, or seats in a theater: a small hall or waiting room.

Lodge. A small house in a park.

Loft. The highest room in a house, particularly if in the roof; also, a gallery raised up in a church to contain the rood, the organ, or singers.

Loggia. An outside gallery or portico above the ground, and contained within the building.

Loop-hole. An opening in the wall of a building, very narrow on the outside, and splayed within, from which arrows or darts might be discharged on an enemy. They are often in the form of a cross, and generally have round holes at the ends.

Lombard Architecture. A name given to the round-arched architecture of Italy, introduced by the conquering Goths and Ostrogoths, and which superseded the Romanesque. It reigned between the eighth and twelfth centuries, during the time that the Saxon and Norman styles were in vogue in England, and corresponded with them in its development into the Continental Gothic.

Lotus. A plant of great celebrity amongst the ancients, the leaves a blossoms of which generally form the capitals of Egyptian columns.

Louver. A kind of vertical window, frequently in the peaks of gables, and the top of towers, and provided with horizontal slats which permit ventilation and exclude rain.

Lozenge Molding. A kind of molding used in Norman architecture, of many different forms, all of which are characterized by lozenge-shaped ornaments.

Lunette. The French term for the circular opening in the groining of the lower stories of towers, through which the bells are drawn up.

LOZENGE MOLDING

LOUVER WIND

Machicolation. A parapet or gallery projecting from the upper part of the wall of a house or fortifi tion, supported by brackets or corbels, and perforated in the lower part so th the defenders of the building might throw down darts, stones, and sometim hot sand, molten lead, etc., upon their assailants below.

Man-hole. A hole through which a man may creep into a drain, cesspo steam-boiler, etc.

Manor-house. The residence of the suzerain or lord of the manor; in Fra the central tower or keep of a castle is often called the *manoir*.

Mansard Roof. Curb roof, invented by François Mansard, a distinguish French architect, who died in 1666.

Mansion. A residence of considerable size and pretension.

Mantel. The work over a fireplace in front of a chimney; especially, a sh usually ornamented, above the fireplace.

Marquetry. Inlaid work of fine hard pieces of wood of different colors, also of shells, ivory, and the like.

Mausoleum. A magnificent tomb or sumptuous sepulchral monument.

Medallion. Any circular tablet on which are embossed figures or busts.

MACHICOLATIO

Mediæval Architecture. The architecture of England, France, Germany, etc., during the Middle Ages, including the Norman a Early Gothic styles. It comprises also the Romanesque, Byzantine and Sa cenic, Lombard, and other styles.

Members. The different parts of a building, the different parts of an enta lature, the different moldings of a cornice, etc.

Merlon. That part of a parapet which lies between two embrasures.

Metope. The square recess between the triglypns in a Doric frieze. It is sometimes occupied by sculptures.

Mezzanine. A low story between two lofty ones. It is called by the French entresol, or inter-story.

Mezzo-rilievo. Or mean relief, in comparison with alto-rilievo, or high relief.

METOPE.

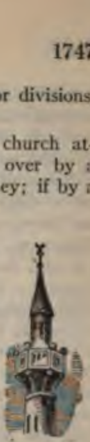
ret. Turkish: a circular turret rising by different stages or divisions, which has a balcony.

ter. Probably a corruption of monasterium — the large church at- to any ecclesiastical fraternity. If the latter be presided over by a it is generally called a Cathedral; if by an abbot, an Abbey; if by a Priory.

te. The sixtieth part of the lower diameter of a column; it easure used by architects to determine the proportions of .

rere. A seat in a stall of a large church made to turn up rd support to a person in a position between sitting and . The under side is generally carved with some ornament, often with grotesque figures and caricatures of different

. A molding returned upon itself at right angles is said to In joinery, the ends of any two pieces of wood of correspond- , cut off at 45°, necessarily abut upon one another so as to ight angle, and are said to miter.



MINARET

llion. So called because of its arrangement in regulated distances; the block or horizontal bracket generally found e cornice of the Corinthian entablature. Less ted, it is sometimes used in the Ionic.

MODILLION

dle. This is a term which has been generally architects in determining the relative propor- the various parts of a columnar ordinance. i-diameter of the column at its base is the which being divided into thirty parts called minutes, any part of the tion is said to be of so many modules and minutes, or minutes alone, in readth, or projection. The whole diameter is now generally preferred dule, it being a better rule of proportion than its half.

stery. A set of buildings adapted for the reception of any of the vari- rs of monks, the different parts of which are described in the separate *Abbey*.

triglyph. The intercolumniations of the Doric order are determined by ber of triglyphs which intervene, instead of the number of diameters of mn, as in other cases; and this term designates the ordinary intercolum- of one triglyph.

ment. A name given to a tomb, particularly to those fine structures in the walls of mediæval churches.

ic. Pictorial representations, or ornaments, formed of small pieces of arble, or enamel of various colors. In Roman houses the floors are often of mosaic, the pieces being cubical. The best examples of mosaic work d in St. Mark's, at Venice.

ue. A Mahometan temple, or place of worship.

ing. When any work is wrought into long regular channels or projec- rming curves or rounds, hollows, etc., it is said to be molded, and each member is called a molding. In mediæval architecture the principal s are those of the arches, doors, windows, piers, etc. In the Early style, the moldings *for some time, formed groups set back in squares, uently very deeply undercut. The scroll molding is also common.*

Small fillets now become very frequ
in section to the bottom of a ship;
each side, like two wings. Later
style the moldings are more varied
hollows and rounds still prevail. T
not so deep, fillets abound, ogees a
and the wave mold, double ogee, or
often seen. In many places the str
a round, the lower half of which is
chamfer. The moldings in the la
degree resemble those of the Decor
extended; they run more into on
fewer fillets, and being, as it w
One of the principal features of
substitution of one, or perhaps t
very large hollows in the set of mo
lows are neither circular nor ellip
like an egg cut across, so that
than the other. The brace mold
bead, where the two ogees meet.
molding, which has been called a
weatherings.

Moldings, Ornamented. Th
seem to have been much enriched,
are remarkable for a profusion of
what is called the zigzag. This s
meander or fret was to the Grec
Saxons, as it is very frequently f
lozenges, crescents, billets, heads of
these, battlements, cables; large ro
as our sailors say, "wormed"; sca
quaint stiff foliages, beaks of birds,
ceivable kind, are sculptured in N
profusion as has been attempted i
English moldings are chiefly the
teristics of this style, though it is t
generally placed in a deep hollow
shadow in the hollow contrasting in
moldings. In this period and in t
ticularly if they are double doors, is
period resemble the former, except
tooth gives way to the ball-flower.
with rosettes set at intervals, which
as the ball-flowers are frequently.
labels of windows are often found i
dicular period the moldings are o
leaved flowers set at intervals, but
are running patterns of vine leave
by old writers are called "vignett
generally called the Tudor leaf.
the same character.

Mullion, Munion. The per
umns, sometimes like slender p

or screen-work from each other. In all styles, in less important work, the mullions are often simply plain chamfered, and more commonly have a very flat hollow on each side. In larger buildings there is often a bead or boutell on the edge, and often a single very small column with a capital. As tracery grew richer, the windows were divided by a larger order of mullion, between which came a lesser or subordinate set of mullions, which ran into each other. The term is also applied to a wood or iron division between two windows.

Multifoil. A leaf ornament consisting of more than five divisions, applied to foils in windows.

Mutule. The rectangular impending block under the corona of the Doric cornice, from which guttæ or drops depend. Mutule is equivalent to modillion but the latter term is applied more particularly to enriched blocks or brackets, such as those of Ionic and Corinthian entablatures.

Narthex. The long arcaded porch forming the entrance into the Christian basilica. Sometimes there was an inner narthex, or lobby, before entering the church. When this was the case, the former was called exo-narthex, and the latter eso-narthex. In the Byzantine churches this inner narthex forms part of the solid structure of the church, being marked off by a wall or row of columns, whereas in the Latin churches it was usually formed only by a wooden or other temporary screen.

Natural Beds. In stratified rocks, the surface of a stone as it lies in the quarry. If not laid in walls in their natural bed the laminæ separate.

Nave. The central part between the arches of a church, which formerly was separated from a chancel or choir by a screen. It is so called from its fancied resemblance to a ship. In the nave were generally placed the pulpit and font. In continental Europe it often also contains a high altar, but this is of rare occurrence in England.

Necking. The annulet or round, or series of horizontal moldings, which separate the capital of a column from the plain part or shaft.

Newel. In mediæval architecture, the circular ends of a winding staircase which stand over each other, and form a sort of cylindrical column.

Newel Post. The post, plain or ornamented, placed at the first, or lowest step, to receive or start the hand-rail upon.

Niche. A recess sunk in a wall, generally for the reception of a statue. Niches sometimes terminate by a simple label, but more commonly by a canopy, and with a bracket or corbel for the figure, in which case they are often called tabernacles.

Norman Style. Was that species of Romanesque which was practised by the Normans, and which was introduced and fully developed in England after they had established themselves in it. The chief features of this style are plainness and massiveness. The arches, windows, and doorways were semicircular, the pillars were very massive, and often built up of small stones laid like brickwork.

Nosings. The rounded and projecting edges of the treads of a stair, or the edge of a landing.

Obelisk. Lofty pillars of stone, of a rectangular form, diminishing toward the top, and generally ornamented with inscriptions and hieroglyphics among the ancient Egyptians.

Observatory. A building erected on an elevated spot of ground for making astronomical observations.

Octostyle. *A portico of eight columns in front.*

Offsets. When the face of a wall is not one continued surface, but s
horizontal jogs, as the wall grows higher and thinner, the jogs are calle

Ogee. The name applied to a molding, partly a hollow and partly
and derived no doubt from its resemblance to an O placed over a G. I
found in Norman work, and is not very common in Early English. It
quent use in Decorated work, where it becomes sometimes double, and i
wave molding; and later still, two waves are connected with a small be
is then called a brace molding. In ancient MSS. it is called a Ressau

Orchestra. In ancient theaters, where the chorus used to dance; i
theaters, where the musicians sit.

Order. A column with its entablature and stylobate is so called. T
the result of the dogmatic laws deduced from the writings of Vitruvius
been exclusively applied to those arrangements which they were th
warrant.

Oriel Window. Gothic: a projecting angular window, commonly
agonal or pentagonal form, and divided by mullions and transoms into
bays and compartments.

Orthography. A geometrical elevation of a building or other object
it is represented as it actually exists or may exist, and not perspectivel
would appear.

Orthostyle. A columnar arrangement in which the columns are pl
straight line.

Ovolo. Same as *Echinus*.

Pagoda. A name given to temples in India and China.

Palace. The dwelling of a king, prince, or bishop.

Pale. A fence picket, sharpened at the upper end.

Pane. Probably a diminutive of panneau, a term applied to the
pieces of glass in a window; same as *Light*.

Panel. Properly a piece of wood framed within four other pieces of
in the styles and rails of a door, filling up the aperture, but often applie
the whole square frame and the sinking itself; also to the ranges of sur
partments in wainscoting, cornices, corbel tables, groined vaults, ceilin

Pantograph, or **Pentagraph.** An instrument for copying on the sa
enlarged or reduced scale.

Pantry. An apartment or closet in which bread and other provi
kept.

Papier-maché. A hard substance made of a pulp from rags or pa
with size or glue, and molded into any desired shape. Much used for
tural ornaments.

Parapet. A dwarf wall along the edge of a roof, or round a terrace
to prevent persons from falling over, and as a protection to the defende
of a siege. Parapets are either plain, embattled, perforated, or panel
last two are found in all styles except the Norman. Plain parapets a
portions of the wall generally overhanging a little, with coping at th
corbel table below. Embattled parapets are sometimes paneled, bu
pierced for the discharge of arrows, etc. Perforated parapets are
various devices — as circles, trefoils, quatrefoils, and other designs — s
light is seen through. Paneled parapets are those ornamented by a
panels, either oblong or square, and more or less enriched, but are not p
These are common in the Decorated and Perpendicular periods.

:eting. A species of plastering decorated by impressing patterns on it vet. These seem generally to have been made by sticking a number of a board in certain lines or curves, and then pressing on the wet plaster in directions, so as to form geometrical figures. Sometimes these devices relief, and in the time of Elizabeth represent figures, birds, foliages, etc. plastering, commonly adopted for the interior surface of chimneys.

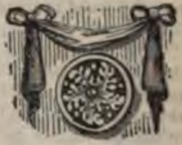
or. A room in a house which the family usually occupy for society and sation, and for receiving visitors. The apartment in a monastery or y where the inmates are permitted to meet and converse with each other, visitors and friends from without.

chial. Belonging or relating to a parish.

uetry, or **Marquetry.** A kind of inlaid floor composed of small pieces l either square or triangular, which are capable of forming, by their dis-, various combinations of figures; this description of joinery is very for the floors of libraries, halls, and public apartments.

y Walls. Partitions of brick or stone between buildings on two adjoining ies.

ra. A circular ornament resembling a dish, often worked in relief on etc.



PATERA

ment. Tessellated, a pavement of mosaic work, the ancients, made of square pieces of stone, etc., Tessera.

lion. A turret or small insulated building, and sed beneath a single roof; also, the projecting front of a building which marks the centre, and sometimes flanks a corner, when it is termed an pavilion.

stal. The square support of a column, statue, d the base or lower part of an order of columns: it consists of a plinth ase, the die, and a talon crowned for a cornice. When the height and re equal, it is termed a square pedestal; one which supports two columns, e pedestal; and if it supports a row of columns without any break, it is ued pedestal.

ment. A low triangular crowning, ornamented, in front of a building, r doors and windows. Pediments are sometimes made in the form of a t; the space enclosed within the triangle is called the tympanum. Also, le ends of classic buildings, where the horizontal cornice is carried across t, forming a triangle with the end of the roof.

ent. A name given to an elongated boss, either molded or foliated, hang down from the intersection of groins, especially in fan tracery, or nd of hammer beams. Sometimes long corbels, under the wall pieces, en so called. The name has also been given to the large masses depend-n enriched ceilings, in the later works of the Pointed style.

ent Posts. A name given to those timbers which hang down the side of rom the plate in hammer beam trusses, and which receive the hammer

entive. A name given to an arch which cuts off, as it were, the corners are building internally, so that the superstructure may become an octagon me. In mediæval *architecture these arches,* when under a spire in the *of a tower, are called Squinches.*

Pendentive Bracketing, or **Cove Bracketing.** Springing from th
tangular walls of an apartment upward to the ceiling, and forming the ho
tal part of the ceiling into a circle or ellipse.

Pentastyle. Having five columns in front.

Pent-roof. A roof with a slope on one side only.

Perch. A measure used in measuring stone work, being 24¾ cu ft an
cu ft, according to locality and custom.

Periptery. An edifice or temple surrounded by a peristyle.

Peristyle. A range of columns encircling an edifice, such as that whic
rounds the cylindrical drum under the cupola of St. Paul's. The colum
Greek peripteral temple form a peristyle also, the former being a circula
the latter a quadrilateral peristyle.

Perpendicular Style. The third and last of the Pointed or Gothic
also called the Florid style.

Perspective Drawing. The art of making such a representation of an
upon a plane surface as shall present precisely the same appearance th
object itself would to the eye situated at a particular point.

Pews. A word of uncertain origin, signifying fixed seats in churches
posed of wood framing, mostly with ornamented ends. They seem to hav
into general use early in the reign of Henry VI, and to have been rent
"well paid for" before the Reformation. Some bench ends are certainl
decorated character, and some have been considered to be of the Early E
period. They are sometimes of plain oak board, two and a half to three
thick, chamfered, and with a necking and finial, generally called a poppy
others are plainly paneled with bold cappings; in others the panels are
mented with tracery or with the linen pattern, and sometimes with r
foliages. The divisions are filled in with thin chamfered boarding, som
reaching to the floor, and sometimes only from the capping to the seat.

Picket. A narrow board, often pointed, used in making fences; a
paling.

Pier-glass. A mirror hanging between windows.

Piers. The solid parts of a wall between windows, and between voids
ally. The term is also applied to masses of brick-work or masonry whi
insulated to form supports to gates or to carry arches, posts, girders, etc.

Pilasters. Are flat square columns, attached to a wall, behind a colu
along the side of a building, and projecting from the wall about a fourt
sixth part of their breadth. The Greeks had a slightly different design f
capitals of pilasters, and made them the same width at top as at bottom, b
Romans gave them the same capitals as the columns, and made them
minished width at the top, similar to the columns.

Pile. A large stake or trunk of a tree, driven into soft ground, as
bottom of a river, or in made land, for the support of a building. (See p.

Pillar, or **Pyller.** A word generally used to express the round or poly
piers, or those surrounded with clustered columns, which carry the main
of a building. Saxon and Early Norman pillars are generally stout cylin
shafts built up of small stones. Sometimes, however, they are quite s
sometimes with other squares breaking out of them (this is more comm
French and German work), sometimes with angular shafts, and sometimes
are plain octagons. In Romanesque Norman work the pillar is some
square, with two or more semicircular or half-columns attached. In th

period the pillars become loftier and lighter, and in most important
gs are a series of clustered columns, frequently of marble, placed side by
metimes set at intervals round a circular centre, and sometimes almost
g each other. These shafts are often wholly detached from the central
hough grouped round it, in which case they are almost always of Purbeck
ersden marbles. In Decorated work the shafts on plan are very often
round a square set anglewise, or a lozenge, the long way down the nave;
tre or core itself is often worked into hollows or other moldings, to show
n the shafts, and to form part of the composition. In this and the latter
the previous style there is generally a fillet on the outer part of the shaft,
what has been called a keel molding. They are also often, as it were,
gether by bands formed of rings of stone and sometimes of metal. The
illars at the jambs of doors and windows, and in arcades, and also those
columns attached to pillars, or standing detached, are generally called

A cylindrical piece of wood, iron, or steel, used to hold two or more
ogether, by passing through a hole in each of them, as in a mortise and
oint, or a pin joint of a truss.

acle. An ornament originally forming the cap or crown of a buttress or
urret, but afterwards used on parapets at the corners of
and in many other situations. It was a weight to counter-
thrust of the groining of roofs, particularly where there
ing buttresses; it stopped the tendency to slip of the stone
of the gables, and counterpoised the thrust of spires; it
the piers to steady the elegant perforated parapets of
riods; and in France, especially, served to counterbalance
ight of overhanging corbel tables, huge gargoyles, etc. In
rly Engish period the smaller buttresses frequently finished
blets, and the more important with pinnacles supported
ustered shafts. At this period the pinnacles were often
ed on these shafts alone, and were open below; and in
ork in this and the subsequent periods they frequently form
and contain statues. In France, pinnacles, like spires,
have been in use earlier than in England. There are small
es at the angles of the tower in the Abbey of Saintes. At
there are pinnacles in a similar position, each composed of
nall shafts, with caps and bases surmounted with small
dal spires. In all these examples the towers have semicircular headed
s.

PINNACLE

of a Roof. The proportion obtained by dividing the height by the
thus, we speak of its being one-half, one-third, one-fourth. When the
of the rafters is equal to the breadth of the building it is denominated

ing-piece. A horizontal timber, with one of its ends wedged into the
the top of a flight of stairs, to support the upper end of the rough strings.

. An open piece of ground surrounded by buildings, generally decorated
tatue, column, or other ornament.

A horizontal geometrical section of the walls of a building; or indi-
on a horizontal plane, of the relative positions of the walls and partitions,
various openings, such as windows and doors, recesses and projections,
s and chimney-breasts, columns, pilasters, etc. This term is often
ly used in the sense of Design.

side of rubble walls, and often of wood framing, it was used ornamented in patterns outside, it is called pargeting.

Plate. The piece of timber in a building which supp rafters.

Plinth. The square block at the base of a column or pede term plinth is applied to the projecting base or water table, g of the first floor.

Plumb. Perpendicular; that is, standing according to a post of a house or wall is plumb.

Plumbing. The lead and iron pipes and other apparat veying water, and for toilet purposes in a building; original and working in lead.

Ply. Used to denote the number of thicknesses of roofin four ply, etc.

Podium. A continued pedestal; a projection from a wal gallery.

Polytriglyph. An intercolumniation in the Doric order triglyphs.

Poppy Heads. Probably from the French *poupèe:* the f ments which terminate the tops of bench ends, either to p stalls. They are sometimes small human heads, sometimes carved images, knots of foliage, or finials, and sometimes de-lis simply cut out of the thickness of the bench end and fered.

Porch. A covered erection forming a shelter to the en door of a large building. The earliest known are the long a porches in front of the early Christian basilicas, called Na In later times they assume two forms—one, the projecting e covering the entrance at the west front of cathedrals, and d into three or more doorways, etc.; and the other, a kind of c chambers open at the ends, and having small windows at th ing as it projects from or recedes within the building, and is further designated by the number of columns its front may consist of.

Post. Square timbers set on end. The term is especially applied to those which support the corners of a building, and are framed into bressummers or crossbeams under the walls.

Posticum. A portico behind a temple.

Presbytery. A word applied to various parts of large churches in a very ambiguous way. Some consider it to be the choir itself; others, what is now named the sacrarium. Traditionally, however, it seems to be applied to the vacant space between the back of the high altar and the entrance to the lady-chapel, as at Lincoln and Chichester; in other words, the back- or retro-choir.

Priming. The laying on of the first shade of color, in oil paint, and generally consisting mostly of oil, to protect and fill the wood.

Priory. A monastic establishment, generally in connection with an abbey, and presided over by a prior, who was a subordinate to the abbot, and held much the same relation to that dignitary as a dean does to a bishop.

Profile. The outline; the contour of a part, or the parts composing an order, as of a base, cornice, etc.; also, the perpendicular section. It is in the just proportion of their profiles that the chief beauties of the different orders of architecture depend. The ancients were most careful of the profiles of their moldings.

Proscenium. The front part of the stage of ancient theaters, on which the actors performed.

Prostyle. A portico in which the columns project from the building to which it is attached.

Protractor. A mathematical instrument for laying down and measuring angles on paper, used in drawing or plotting.

Pseudo-dipteral. False double-winged. When the inner row of columns of a dipteral arrangement is omitted and the space from the wall of the building to the columns is preserved, it is pseudo-dipteral.

Puddle. To settle loose dirt by turning on water, so as to render it firm and solid.

Pugging. A coarse kind of mortar laid on the boarding, between floor joists, to prevent the passage of sound; also called deafening.

Pulpit. A raised platform with enclosed front, whence sermons, homilies, etc., were delivered. Pulpits were probably derived in their modern form from the ambones in the early Christian church. There are many old pulpits of stone, though the majority are of wood. Those in the churches are generally hexagonal or octagonal; and some stand on stone bases, and others on slender wooden stems, like columns. The designs vary according to the periods in which they were erected, having paneling, tracery, cuspings, crockets, and other ornaments then in use. Some are extremely rich, and ornamented with color and gilding. A few also have fine canopies or sounding boards. Their usual place is in the nave, mostly on the north side, against the second pier from the chancel arch. Pulpits for addressing the people in the open air were common in the Mediæval period, and stood near a road or cross. Thus, there was one at Spitalfields, and one at St. Paul's, London. External pulpits still remain at Magdalen College, Oxford, and at Shrewsbury, England.

Purlins. Those pieces of timbers which support the rafters to prevent them from sinking.

Putlog. Horizontal pieces for supporting the floor of a scaffo inserted into putlog holes, left for that purpose in the masonry

Putty in Plastering. Lump lime slacked with water to th cream, and then left to harden by evaporation till it becomes lik is then mixed with plaster of Paris, or sand, for the finishing c

Puzzolana. A grayish earth used for building under water.

Pyramid. A solid, having one of its sides, called a base, a the other sides triangles, these points joining in one point at th vertex. Pyramids are called triangular, square, etc., accordin their bases.

Pyx. In Roman Catholic churches, the box in which the ho wafer, is kept.

Quadrangle. A square or quadrangular court surrounded was often done formerly in monasteries, colleges, etc.

Quarry. A pane of glass cut in a diamond or lozenge form

Quarry-face. Ashlar as it comes from the quarry, squared only, with split face. In distinction from Rock-face, in that weather-worn, while Quarry-face should be fresh split. The used indiscriminately.

Quatrefoil. Any small panel or perforation in the form of a f Sometimes used alone, sometimes in circles and over the aisle w frequently in square panels. They are generally cusped, and t feathered.

Queen Truss. A truss framed with two vertical tie-posts, i the king-post, which has but one. The upright ties are called

Quirk Moldings. The convex part of Grecian moldings wh the top, forming a reëntrant angle, with the surface which cov

Quoins. Large squared stones at the angles of buildings, generally used to stop the rubble or rough stone work, and tha be true and stronger. Saxon quoin stones are said to have b one long and one short stone alternately. Early quoins are axed; in later times they had a draught tooled by the chisel edges, and later still were worked fine from the saw.

Rafters. The joist to which the roof boarding is nailed. are the upper timbers in a truss, having the same inclination rafters.

Rail. A piece of timber or metal extending from one post fences, balustrades, staircases, etc. In framing and panelin pieces are called rails, and the perpendicular, *stiles*.

Raking. Moldings whose arrises are inclined to the horizo

Ramp. A concavity on the upper side of hand railings fo made by a sudden rise of the steps above. Any concave bend c

Work. Ashlar laid in horizontal courses; same as coursed ashlar.

e. A groove on the edges of a board.

s. A depth of some inches in the thickness of a wall, as a niche, etc.

tory. The hall of a monastery, convent, etc., where the religious took
ef meals together. It much resembled the great halls of mansions,
tc., except that there frequently was a sort of ambo, approached by
m which to read the Legenda Sanctorum, etc., during meals.

t. A flat, narrow molding, used to separate from each other the parts
ers of compartments and panels, to form frets, knots, etc.

ssance (a new birth). A name given to the revival of Roman architec-
h sprang into existence in Italy as early as the beginning of the fifteenth
and reached its zenith in that country at the close of the century.
e several divisions of this style as developed in different localities; viz.,
orentine Renaissance, of which the Pitti Palace, by Brunelleschi, is one
st examples.
netian Renaissance, characterized by its elegance and richness.
oman Renaissance, which originated in Rome, under the architects
Bronte, Vignola, and Michael Angelo. Of this style the Farnese Palace,
's, and the modern Capitol at Rome are the best examples.
ench Renaissance, introduced into France in the latter part of the fif-
ntury, by Italian architects, where it flourished until the middle of the
th century. The Renaissance style was introduced into Germany
e middle of the sixteenth century, and into England about the same
John of Padua, architect to Henry VIII. This style in England is gen-
own under the name of Elizabethan.

ring. In drawing, finishing a perspective drawing in ink or color, to
the spirit and effect of the design. The first coat of plaster on brick
work.

os, Dorsal, or **Dossel.** The screen or other ornamental work at the
n altar. In some large English cathedrals, as Winchester, Durham, St.
tc., this is a mass of splendid tabernacle work, reaching nearly to the
In smaller churches there are sometimes ranges of arcades or panelings
e altars; but, in general, the walls at the back and sides of them were of
onry, and adorned with hangings or paraments. In the large churches
ental Europe the high altar usually stands under a sort of canopy or
and the sacrarium is hung round at the back and sides with curtains on
rods.

lated Work. That in which the courses are arranged in a form like
es of a net. The stones or bricks are square and placed lozenge-wise.

. The continuation of a molding, projection, etc., in an opposite

Head. One that appears both on the face and edge of a work.

l. The two vertical sides of an aperture, between the front of a wall and
w or door frame.

A molding or projecting piece upon the interior of a vault, or used to
ery and the like. The earliest groining had no ribs. In early Norman
in flat arches crossed each other, forming ogive ribs. These by degrees
arrower, had greater projection, and were chamfered. In later Nor-
k the ribs were often formed of a large roll placed upon the flat band,
of two rolls side by side with a smaller roll or a fillet between them,

much like the lower member. Sometimes they are enriched with
other Norman decorations, and about this time bosses became of
use. As styles progressed, the moldings were more undercut, rich
elaborate, and had the dog-tooth or ball-flower or other characteris
in the hollows. In all instances the moldings are of similar cont
of arches, etc., of the respective periods. Later, wooden roofs are
into cants or polygonal barrel vaults, and in these the ribs are gener
of rounds, and form square or stellar panels, with carved bosses or
intersections.

Ridge. The top of a roof which rises to an acute angle.

Ridge-pole. The highest horizontal timber in a roof, extendin
top of the several pairs of rafters of the trusses, for supporting the
jack rafters.

Rilievo, or **Relief.** The projection of an architectural ornamen

Rise. The distance through which anything rises, as the rise
inclined plane.

Riser. The vertical board under the tread in stairs.

Rococo Style. A name given to that variety of the Renaissance
vogue during the seventeenth and the latter part of the sixteenth

Romanesque Style. The term Romanesque embraces all thos
chitecture which prevailed between the destruction of the Roman E
beginning of Gothic architecture. In it are included the Early Ro
architecture, Byzantine, Mahometan, and the later Romanesqu
proper, which was developed in Italy, France, England, and Ge
later Romanesque, which was quite different from the preceding, ca
during the tenth century, and reached its height during the twelfth
in the thirteenth century gave way to the Pointed or Gothic style.
Romanesque architecture is known under the name of the Saxon,
Lombard styles, according to the different political periods.

Rood. A name applied to a crucifix, particularly to those whic
in the rood-loft or chancel screens. These generally had not only
the crucified Saviour, but also those of St. John and the Virgin M
one on each side. Sometimes other saints and angels are by them
of the screen is set with candlesticks or other decoration.

Rood-loft, Rood-screen, Rood-beam, Jube Gallery, etc.
ment to carry the crucifix or rood, and to screen off the chancel fr
the church during the breviary services, and as a place whence t
parts of those services. Sometimes the crucifix is carried simply on
verse beam, with or without a low screen, with folding-doors below b
part of such support. In European churches the general constructi
screens is close paneling beneath, about 3 feet to 3 feet 6 inches h
stands screen work composed of slender turned balusters or re
mullions, supporting tracery more or less rich, with cornices, crest
often painted in brilliant colors and gilded. These not only enclose
but also chapels, chantries, and sometimes even tombs. In Engl
and some private houses, the great halls were screened off by a low p
end opposite to the dais, over which was a gallery for the use of
spectators. These screens were sometimes close and sometimes gl

Rood-tower. A name given by some writers to the central t
over the intersection of the nave and chancel with the transepts.

The covering or upper part of any building.

ıg. The material put on a roof to make it water-tight.

Window. A name given to a circular window with radiating tracery; ıo wheel window.

ım. An elevated platform from which a speaker addresses an audience.

da. A building which is round both within and without. A circular ler a dome in large buildings is also called the rotunda.

ıcast. A sort of external plastering in which small sharp stones are nd which, when wet, is forcibly thrown or cast from a trowel against the which it forms a coating of pleasing appearance. Roughcast work has d in Europe for several centuries, where it was much used in timber nd when well executed the work is sound and durable. The mortar for t work should always have cement mixed with it.

e Work. Masonry of rough, undressed stones. When only the rough-ılarities are knocked off, it is called scabbled rubble, and when the stones course are rudely dressed to nearly a uniform height, ranged rubble.

ıture. The figure of a rope or staff, which is frequently used to fill up ıgs of columns, the convexity of which contrasts with the concavity of ıgs, and serves to strengthen the edges. Sometimes, instead of a convex ıe flutings are filled with a flat surface; sometimes they are ornamentally ınd sometimes on pilasters, etc. Rudentures are used in relief without as their use is to give greater solidity to the lower part of the shaft, and ıe edges. They are generally only used in columns which rise from the ınd are not to reach above one-third of the height of the shaft.

c or **Rock Work.** A mode of building in imitation of nature. This applied to those courses of stone work the face of which is jagged or as to present a rough surface. That work is also called rustic in which ontal and vertical channels are cut in the joinings of stones, so that when gether an angular channel is formed at each joint. *Frosted rustic work* nargins of the stones reduced to a plane parallel to the plane of the wall, mediate parts having an irregular surface. *Vermiculated rustic work* has ermediate parts so worked as to have the appearance of having been worms. *Rustic chamfered work,* in which the face of the stones is and parallel to the face of the wall, and the angles beveled to an angle of dred and thirty-five degrees with the face so that two stones coming on the wall, the beveling will form an internal right angle.

ty. A small chamber attached to churches, where the chalices, vest-ooks, etc., were kept by the officer called the sacristan. In the early basilicas there were two semicircular recesses or apsides, one on each ıe altar. One of these served as a sacristy, and the other as the biblio-library. Some have supposed the sacristy to have been the place where nents were kept, and the vestry that where the priests put them on; but rom Durandus that the sacrarium was used for both these purposes. es the place where the altar stands enclosed by the rails has been called n.

e Bars. Narrow horizontal iron bars passing from mullion to mullion, through the whole window, from side to side, to steady the stone work, rm stays, to which the lead work is secured. When the bays of the are wide, the lead lights are further strengthened by upright bars *hrough eyes forged on the saddle bars,* and called stanchions. When

saddle bars pass right through the mu
jambs, they have sometimes been call

Sagging. The bending of a body i
upon it.

Salient. A projection.

Salon. A spacious and elegant ap
for state purposes, or for the recep
through two stories of the house.
circular.

Sanctuary. That part of a churc
sacred or retired part of a temple. A

Sanctus Bell-cot, or **Turret.** A
sounded at various parts of the servic
etc., are read. This differs but little
generally on the top of the arch dividi
however, the bell seems to have been
land sanctus bells have also been pla
nental Europe they run up into a s
France, and *guglio* in Italy.

Saracenic Architecture. That E
which distributed itself over the wor
modification and combination of the
conquered.

Sarcophagus. A tomb or coffin n
body.

Sash. The framework which hold

Scabble. To dress off the roughe
with a stone axe or scabbling hamme

Scagliola. An imitation of colore
bination of gypsum, glue, isinglass, an
polish, invented between 1600 and 16

Scantling. The dimensions of a
also, studding for a partition, when u

Scarfing. The joining and boltin
versely, so that the two appear as on

Sconce. A fixed hanging or proje

Scotia. A concave molding, most
deep shadow on itself, and is thereb
as in a base. It is like a reversed ov
would present.

Scratch Coat. The first coat of p
for the second coat.

Screeds. Long narrow strips of pl
carefully faced out of wind, to serve
between them.

Screen. Any construction subdivi
as a choir, chantry, chapel, etc. The
shutting off the chorus cantantium in
cancelli enclosing the bema, altar, an
chief screens in a church are those

ry services are recited. In Continental Europe this is done not only by screen work, but also, when these are of open work, by curtains, the ng no part in these services. In England screens were of two kinds: en wood-work, generally called rood-screens or jubes, and which the ll *grilles, clôtures du chœur;* the other, massive enclosures of stone work ith niches, tabernacles, canopies, pinnacles, statues, crestings, etc., as bury, York, Gloucester, and many other places.

g. Fitting wood-work to an irregular surface.

. A drawing showing the internal heights of the various parts of a It supposes the building to be cut through entirely, so as to exhibit the heights of the internal doors and other apertures, the heights of , thicknesses of the floors, etc. It is one of the species of drawings to the exhibition of a Design.

. Seats used by the celebrants during the pauses in the mass. They lly three in number — for the priest, deacon, and sub-deacon — and gland almost always a species of niches cut into the south walls of separated by shafts or by a species of mullions, and crowned with pinnacles, and other enrichments more or less elaborate. The piscina sometimes are attached to them. In Continental Europe the sedilia movable seats; a single stone seat has rarely been found.

The horizontal line shown where a wall is reduced in thickness, and, tly, the part of the thicker portion appears projecting before the In plinths this is generally simply chamfered. In other parts of work is generally concealed by a projecting string. Where, as in parapets, part projects before the lower, the break is generally hid by a corbel e portions of buttress caps which recede one behind another are also offs.

In Classical architecture that part of a column between the necking ophyge at the top of the base. In later times the term is applied to lumns either standing alone or in connection with pillars, buttresses, ulting, etc.

oof, or **Lean-to.** A roof with only one set of rafters, falling from a lower wall, like an aisle roof.

A piece of timber placed in an oblique direction to support a building nporarily while it is being repaired or altered.

A sort of ark or chest to hold relics. It is sometimes merely a small rally with a raised top like a roof; sometimes an actual model of sometimes a large construction, like that of Edward the Confessor at er, of St. Genevieve at Paris, etc. Many are covered with jewels in way; that of San Carlo Borromeo, at Milan, is of beaten silver.

Are the timbers on the ground which support the posts and superstruc- imber building. The term is most frequently applied to those pieces or stone at the bottom of doors or windows.

ck. The inclined stone from which an arch springs.

gs. The narrow boards which form a plinth around the margin of a generally called the base.

. A piece of timber laid on the ground to receive floor joists.

The lower horizontal face of anything as, for example, of an entab- ing on and lying open between the columns, or the under face of an arch *hickness is seen.*

Sound Board. The covering of a pulpit to deflect the sound into a churc

Spall. Bad or broken brick; stone chips.

Span. The distance between the supports of a beam, girder, arch, truss,

Spandrel, or **Spandril.** The space between any arch or curved brace and level label, beams, etc., over the same. The spandrels over doorways in Per dicular works are generally richly decorated.

Specification. Architect's. The designation of the kind, quality, quantity of work and material to go in a building, in conjunction with the worl drawings.

Spire. A sharply pointed pyramid or large pinnacle, generally octagon England, and forming a finish to the tops of towers. Timber spires are common in England. Some are covered with lead in flat sheets, others with same metal in narrow strips laid diagonally. Very many are covered shingles. In Continental Europe there are some elegant examples of spir open timber work covered with lead.

Splayed. The jamb of a door, or anything else of which one side make oblique angle with the other.

Springer. The stone from which an arch springs; in some cases this capital, or impost; in other cases the moldings continue down the pier. lowest stone of the gable is sometimes called a springer.

Squinches. Small arches or corbeled set-offs running diagonally and, were, cutting off the corners of the interior of towers, to bring them from square to the octagon, etc., to carry the spire.

Squint. An oblique opening in the wall of a church; especially, in medi architecture, an opening so placed as to afford a view of the high altar from transept or aisles.

Staging. A structure of posts and boards for supporting workmen material in building.

Stall. A fixed seat in the choir for the use of the clergy. In early Chris times the thronus cathedra, or seat of the bishop, was in the center of the or bema behind the altar, and against the wall; those of the presbyters also against the wall, branching off from side to side around the semicircle. In times the stalls occupied both sides of the choir, return seats being placed at ends for the prior, dean, precentor, chancellor, or other officers. In genera cathedrals, each stall is surmounted by tabernacle work, and rich cano generally of oak.

Stanchion. A word derived from the French *étançon*, a wooden post, ap to the upright iron bars which pass through the eyes of the saddle bars or zontal irons to steady the lead lights. The French call the latter *traverses*, stanchions *montants*, and the whole arrangement *armature*. Stanchions quently finish with ornamental heads forged out of the iron.

Steeple. A general name for the whole arrangement of tower, belfry, spire,

Stereobate. A basement, distinguished from the nearly equivalent term lobate by the absence of columns.

Stile. The upright piece in framing or paneling.

Stilted. Anything raised above its usual level. An arch is stilted whe centre is raised above the line from which the arch appears to spring.

Stoop. A seat before the door; often a porch with a balustrade and seat the sides.

Stoup. A basin for holy water at the entrance of Roman Catholic churches, into which all who enter dip their fingers and cross themselves.

Straight Arch. A form of arch in which the intrados is straight, but with its joints radiating as in a common arch.

Strap. An iron plate for connecting two or more timbers, to which it is screwed by bolts. It generally passes around one of the timbers.

Stretcher. A brick or block of masonry laid lengthwise of a wall.

String Board. A board placed next to the well-hole in wooden stairs, terminating the ends of the steps. The string piece is the piece of board put under the treads and risers for a support, and forming the support of the stair.

String-course. A narrow, vertically faced and slightly projecting course in an elevation. If window-sills are made continuous, they form a string-course; but if this course is made thicker or deeper than ordinary window-sills, or covers a set-off in the wall, it becomes a blocking-course. Also, horizontal moldings running under windows, separating the walls from the plain part of the parapets, dividing towers into stories or stages, etc. Their section is much the same as the labels of the respective periods; in fact, these last, after passing round the windows, frequently run on horizontally and form strings. Like labels, they are often decorated with foliages, ball-flowers, etc.

Studs, or **Studding.** The small timbers used in partitions and outside wooden walls, to which the laths and boards are nailed.

Style. The term style in architecture has obtained a conventional meaning beyond its simpler one, which applies only to columns and columnar arrangements. It is now used to signify the differences in the moldings, general outlines, ornaments, and other details which exist between the works of various nations, and also those differences which are found to exist between the works of any nation at different times.

Stylobate. A basement to columns. Stylobate is synonymous with pedestal, but is applied to a continued and unbroken substructure or basement to columns, while the latter term is confined to insulated supports. The Greek temples generally had three or more steps all around the temple, the base of the column resting on the top step; this was the stylobate.

Subsellium. A name sometimes given to the seat in the stalls of churches; same as miserere.

Summer. A girder or main-beam of a floor; if supported on two-story posts and open below, it is called a Brace-summer.

Surbase. A cornice or series of moldings on the top of the base of a pedestal, podium, etc.; a molding above the base.

Surface. To make plane and smooth.

Systyle. An intercolumniation to which two diameters are assigned.

Tabernacle. A species of niche or recess in which an image may be placed. They are generally highly ornamented and often surmounted with crocketed gables. The word tabernacle is also often used to denote the receptacle for relics, which was often made in the form of a small house or church.

Tabernacle Work. The rich ornamental tracery forming the canopy, etc., to a tabernacle, is called tabernacle work; it is common in the stalls and screens of cathedrals, and in them is generally open or pierced through.

Tail Trimmer. A trimmer next to the wall, into which the ends of joists are fastened to avoid *flues.*

Tamp. To pound the earth down around a wall after it has been throw

Tapestry. A kind of woven hangings of wool or silk, ornamented with fi and used formerly to cover and adorn the walls of rooms. They were oft the most costly materials and beautifully embroidered.

Temple. An edifice destined, in the earliest times, for the public exerc religious worship.

Templet, or **Template.** A mold used by masons for cutting or setting A short piece of timber sometimes laid under a girder.

Terminal. Figures of which the upper parts only, or perhaps the hea shoulders alone, are carved, the rest running into a parallelopiped, and sometimes into a diminishing pedestal, with feet indicated below, or even without them, are called terminal figures.

ANCIENT TERMIN

Terra-cotta. Baked clay of a fine quality. Much used for bas-reliefs for adorning the friezes of temples. In modern times employed for architectural ornaments, statues, vases, etc.

Tessellated Pavements. Those formed of tesseræ, or, as some write it, tessellæ, or small cubes from half an inch to an inch square, like dice, of pottery, stone, marble, enamel, etc.

Tetrastyle. A portico of four columns in front.

Tholobate. That on which a dome or cupola rests. This is a term not in general use, but it is not the less of useful application. What is generally termed the attic above the peristyle and under the cupola of St. Paul's, Lo would be correctly designated the tholobate. A tholobate of a different de tion, and one to which no other name can well be applied, is the circular structure to the cupola of the University College, London.

Throat. A channel or groove made on the under-side of a string-c coping, etc., to prevent water from running inward toward the walls.

Tie. A timber, rod, chain, etc., binding two bodies together, which h tendency to separate or diverge from each other. The *tie-beam* connect bottom of a pair of principal rafters, and prevents them from bursting o wall.

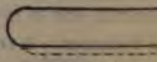
Tiles. Flat pieces of clay burned in kilns, to cover roofs in place of sla lead. Also, flat pieces of burned clay, either plain or ornamented, gla unglazed, used for floors, wainscoting, and about fireplaces, etc. Small s pieces of marble are also called tile.

Tongue. The part of a board left projecting, to be inserted into a gro

Tooth Ornament. One of the peculiar marks of the Early English per Gothic architecture, generally inserted in the hollow moldings of doo windows, etc.

Torso. A mutilated statue of which nothing remains but the trunk. Co with twisted shafts have also this term. Of this kind there are several va

Torus. A protuberance or swelling, a molding whose form is convex, and generally nearly approaches a semicircle. It is most frequently used in bases, and is generally the lowest molding in a base.



TORUS

r. An elevated building originally designed for purposes of defence.
ildings are of the remotest antiquity, and are, indeed, mentioned in the
Scriptures. In mediæval times they were generally attached to churches,
eries, to castles, or used as bell-towers in public places of large cities.
hes, the towers of the Saxon period were generally square. Norman
were also generally square. Many were entirely without buttresses;
ad broad, flat, shallow projections which served for this purpose. The
ndows were very narrow, with extremely wide splays inside, probably in-
o be defended by archers. The upper windows, like those of the preced-
, were generally separated into two lights, but by a shaft or short column,
by a baluster. Early English towers were generally taller, and of more
proportions. They almost always had large projecting buttresses, and
ly stone staircases. The lower windows, as in the former style, were
ly mere arrow-slits; the upper were in couplets or triplets, and sometimes
r top had an arcade all around. The spires were generally broach spires;
etimes the tower tops finished with corbel courses and plain parapets, and
with pinnacles. There are a few Early English towers which break into
gon from the square toward the top, and still fewer which finish with two
Both these methods of termination, however, are common in Continental
At Vendôme, Chartres, and Senlis the towers have octagonal upper
urrounded with pinnacles, from which elegant spires arise. In the North
and in Rome, they are generally tall square shafts in four to six stages,
buttresses, with couplets or triplets of semicircular windows in each
enerally crenellated at top, and covered with a low pyramidal roof. The
own leaning tower at Pisa is cylindrical, in five stories of arcaded colon-
In Ireland there are in some of the churchyards very curious round

ery. The ornamental filling in of the heads of windows, panels, circular
s, etc., which has given such characteristic beauty to the architecture of
teenth century. Like almost everything connected with mediæval archi-
this elegant and sometimes fairy-like decoration seems to have sprung
e smallest beginnings. The circular-headed window of the Normans
ly gave way to the narrow-pointed lancets of the Early English period,
less light was afforded by the latter system than by the former, it was
y to have a greater number of windows; and it was found convenient to
hem together in couplets, triplets, etc. When these couplets were as-
l under one label, a sort of vacant space or spandrel was formed over the
and under the label. To relieve this, the first attempts were simply to
e this flat spandrel, first by a simple lozenge-shaped or circular opening,
erward by a quatrefoil. By piercing the whole of the vacant spaces in
dow head, carrying moldings around the tracery, and adding cusps to it,
nation of tracery was complete, and its earliest result was the beautiful
rical work such as is found at Westminster Abbey.

sept. That portion of a church which passes transversely between the
d choir at right angles, and so forms a cross on the plan.

som. The horizontal construction which divides a window into
or stages. Transoms are sometimes simple pieces of mullions placed
rsely as cross-bars, and in later times are richly decorated with
s, etc.

erse. To plane in a direction across the grain of the wood, as to traverse
by planing across the boards.

d. *The horizontal part of a step of a stair.*

Trefoil. A cusping the outline of which is derived from a three-leaved fl or leaf, as the quatrefoil and cinque-foil are from those with four and five.

Trellis. Lattice-work of metal or wood for vines to run on.

Trestle. A movable frame or support for anything; when made of a piece with four legs it is called by carpenters a horse.

Triforium. The arcaded story between the lower range of piers and ar and the clere-story. The name has been supposed to be derived from *tres fores* — three doors, or openings — that being a frequent number of arch each bay.

Triglyph. The vertically channeled tablets of the Doric frieze are c triglyphs, because of the three angular channels in them — two perfect anc divided — the two chamfered angles or hemiglyphs being reckoned as one. square sunk spaces between the triglyphs on a frieze are called metopes.

Trim. Of a door, sometimes used to denote the locks, knobs, and hing

Trimmer. The beam or floor joist into which a header is framed.

Trimmer Arch. An arch built in front of a fireplace, in the thickness o floor, between two trimmers. The bottom of the arch starting from the chi and the top pressing against the header.

Tuck-pointing. Marking the joints of brickwork with a narrow pa ridge of fine putty.

Tudor Style. The architecture which prevailed in England during the of the Tudors; its period is generally restricted to the end of the reign of H VIII.

Turret. A small tower, especially at the angles of larger buildings, some overhanging and built on corbels, and sometimes rising from the ground.

Tuscan Order. The plainest of the five orders of Classic architecture.

Tympanum. The triangular recessed space enclosed by the cornice bounds a pediment. The Greeks often placed sculptures representing sub connected with the purposes of the edifice in the tympana of temples, as a Parthenon and Ægina.

Under-croft. A vaulted chamber under ground.

Upset. To thicken, and shorten as by hammering a heated bar of iron o end.

Vagina. The upper part of the shaft of a terminus, from which the bu figure seems to rise.

Valley. The internal angle formed by two inclined sides of a roof.

Valley Rafters. Those which are disposed in the internal angle of a ro form the valleys.

Vane. The weathercock on a steeple. In early times it seems to have of various forms, as dragons, etc.; but in the Tudor period the favorite d was a beast or bird sitting on a slender pedestal, and carrying an upright ro which a thin plate of metal is hung like a flag, ornamented in various way

Vault. An arched ceiling or roof. A vault is, indeed, a laterally conj series of arches. The arch of a bridge is, strictly speaking, a vault. Inters vaults are said to be groined. See *Groined Vaulting* for fuller descripti vaults.

Verge. The edge of the tiling, slate or shingles, projecting over the gab roof, that on the horizontal portion being called eaves.

Verge Board. Often corrupted into Barge Board; the board under the verge of gables, sometimes molded, and often very richly carved, perforated, and cusped, and frequently having pendants, and sometimes finials, at the apex.

Vermiculated. Stones, etc., worked so as to have the appearance of having been worked by worms.

VERMICULATED

Vestibule. An anti-hall, lobby, or porch.

Vestry. A room adjoining a church, where the vestments of the minister are kept and parish meetings held. In American Protestant churches, the Sunday-school room is often called the vestry.

Viaduct. A structure of considerable magnitude, and usually of masonry, for carrying a railway across a valley.

Vignette. A running ornament, representing, as its name imports, a little vine, with branches, leaves, and grapes. It is common in the Tudor period, and runs or roves in a large hollow or casement. It is also called Trayle.

Villa. A country house for the retreat of the rich.

Volute. The convolved or spiral ornament which forms the characteristic of the Ionic capital. Volute, scroll, helix, and cauliculus are used indifferently for the angular horns of the Corinthian capital.

Voussoir. One of the wedge-like stones which form an arch; the middle one is called the key-stone.

Wainscot. The wooden lining of walls, generally in panels.

Wall Plates. Pieces of timber which are placed on top of brick or stone walls so as to form the support to the roof of a building.

Warped. Twisted out of shape by seasoning.

Water Table. A slight projection of the lower masonry or brickwork on the outside of a wall a few feet above the ground as a protection against rain.

Weather Boarding. Boards lapped over each other to prevent rain, etc., from passing through.

Weathering. A slight fall on the top of cornices, window-sills, etc., to throw off the rain.

Wicket. A small door opening in a larger. They are common in mediæval doors, and were intended to admit single persons, and guard against sudden surprises.

Wind. A turn, a bend. A wall is *out of wind* when it is a perfectly flat surface.

Wing. A side building less than the main building.

Withes. The partition between two chimney flues in the same stack.

ARCHITECTURAL TERMS AS DEFINED IN VARIOUS BUILDING LAWS

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Terms Defined

[The following terms chance to be defined in sundry building codes, which a mentioned in each case. The fact that other codes are not mentioned is not nec sarily a proof that the term is not also elsewhere in use as defined.]

Adjoining Owner. The owner of the premises adjoining those on whic work is doing or to be done. [*District of Columbia.*]

Alteration. Any change or addition except necessary repairs in, to, or up any building affecting an external, party, or partition wall, chimney, floor, stairway, and "to alter" means to make such change or addition. [*Boston an Denver.*]

Appendages. Dormer-windows, cornices, moldings, bay-windows, towe spires, ventilators, etc. [*Chicago and Minneapolis.*]

Areas. Sub-surface excavations adjacent to the building-line for lighting ventilation of cellars or basements. [*District of Columbia.*]

Attic Story. A story situated either in whole or in part in the roof. [*Dent and District of Columbia.*]

Base. "The *base* of a brick wall" means the course immediately above th foundation wall. [*Cincinnati and Cleveland.*]

Basement Story. One whose floor is 12″ or more below the sidewalk, an whose height does not exceed 12′ in the clear; all such stories that exceed 1 high shall be considered as first stories. [*Chicago and Louisville.*]

A story whose floor is 12″ or more below the grade of sidewalk. [*Milwaukee*

A story whose floor is 3′ or more below the sidewalk, and whose height do not exceed 11′ in the clear; all such stories that exceed 11′ high shall be co sidered as first stories. [*Minneapolis.*]

A story suitable for habitation, partially below the level of the adjoining stre or ground.* [*District of Columbia and Denver.*]

(See **Cellar.**)

Bay-window. A first-floor projection for a window other than a tower-pro jection or show-window. [*District of Columbia.*]

Any projection for a window other than a show-window. [*Denver.*]

Bearing Walls. Those on which beams, trusses, or girders rest. [*New Yor and San Francisco.*]

Brick Building. A building the walls of which are built of brick, stone, iro or other substantial and incombustible materials. [*Boston, Denver, and Kans City.*]

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* And below the first floor of joists. [District of Columbia.]

ding. Any construction within the scope and purview of these regula-
[*District of Columbia.*]

ding Line. The line of demarcation between public and private space.
t of Columbia.]

ding Owner. The owner of premises on which work is doing or to be
[*District of Columbia.*]

ness buildings shall embrace all buildings used principally for business
es, thus including, among others, hotels, theaters, and office-buildings.
o, Louisville, Milwaukee, and Minneapolis.]

ar. Basement or lower story of any building, of which one-half or more
height from the floor to the ceiling is below the level of the street* ad-
.† [*Boston, Denver, and Kansas City.*]
ion of building below first floor of joists, if partially or entirely below the
the adjoining parking, street, or ground, and not suitable for habitation.
t of Columbia.]

ent-mortar. A proper proportion of cement and sand without the ad-
e of lime. [*Kansas City.*]

sion Wall. One that separates part of any building from another part
same building. [*Cincinnati and Cleveland.*]
r-bearing walls extending through buildings from front to rear, and sepa-
stores and tenements in buildings or blocks owned by the same party.
apolis.]

Partition-wall.)

lling-house Class. All buildings except public buildings and buildings
warehouse class. [*Cincinnati and Cleveland.*]
not apply to buildings accommodating more than three families. [*San
sco.*]

rnal Wall. Every outer wall or vertical enclosure of a building other
party-wall. [*Boston, Cincinnati, Cleveland, Denver, District of Columbia,
City, and Providence.*]

t Story. The story the floor of which is at or first above the level of the
lk or adjoining ground, the other stories to be numbered in regular suc-
, counting upward. [*Denver and District of Columbia.*]

ting Course. A projecting course or courses under base of foundation
[*Cincinnati and Cleveland.*]

ndation. That portion of wall below level of street curb,‡ and, where the
not on a street, that portion of wall below the level of the highest ground
the wall. [*Boston, Kansas City, New York, and Providence.*]
ion of exterior wall below surface of adjoining earth or pavement, and
of partition or party wall below level of basement or cellar floor. [*Dis-
Columbia and Denver.*]

ndation, Basement, or Cellar Walls. That part of walls of building that
w the floor or joists, which are on or next above the grade line. [*Detroit.*]
ion of the wall below the level of street curb, in front of the central line of
g. [*San Francisco.*]

ound. [*Providence.*]
d not suitable for habitation. [*Denver.*]
And serve as supports for piers, columns, girders, beams, or other walls."
ork.)

Incombustible Scantling Partition. One plastered on both sides upon i lath or wire cloth, and filled in with brickwork 8″ high from floor, provided building is not over 80′ high. [*Chicago.*]

Incombustible Roofing. Covered with not less than three (3) thickne roofing-felt, and good coat of tar and gravel, or with tin, corrugated-iron, or o fire-resisting material with standing-seam or lap-joint. [*Denver.*]

Lengths. Walls are deemed to be divided into distinct *lengths* by re walls, and the length of every wall is measured from the center of one return to the center of another, provided that such return walls are external or p cross-walls of the thickness herein required, and bonded into the walls so dee to be divided. [*Cincinnati and Cleveland.*]

Inflammable Material. Dry goods, clothing, millinery, and the lik stores, flyings or goods in factories, or other substance readily ignited by d pings or flyings from electric lights. [*Minneapolis.*]

Lodging-house. A building in which persons are temporarily accommod with sleeping[*] apartments, and includes hotels. [*Boston and Kansas City.*]

Any building or portion thereof in which persons are lodged for hire for than a week at one time. [*District of Columbia and Providence.*]

Any building or portion thereof in which persons are lodged for hire ten rarily, and includes hotels. [*Denver.*]

Mansard Roof. One formed with an upper and under set of rafters, upper set more inclined to the horizon than the lower set. [*Denver and Di of Columbia.*]

Oriel Window. A projection for a window above the first floor. [*Di of Columbia.*]

Partition. An interior division constructed of iron, glass, wood, lath plaster, or other destructible natures. [*District of Columbia.*]

Partition-wall. Any interior wall of masonry in a building. [*Bo Kansas City, and Providence.*]

An interior wall of non-combustible material. [*District of Columbia.*]

Any interior division constructed of iron, glass, wood, lath and plaster any combination of those materials. [*Denver.*]

(See **Division Wall.**)

Party-wall. Every wall used, or built, in order to be used, as a separatio two or more buildings.[†] [*Boston, Cincinnati, Cleveland, Denver, Kansas and Providence.*]

A wall built upon dividing line between adjoining premises for their com use. [*District of Columbia.*]

Parking. The space between the sidewalk and the building line. [*Di of Columbia.*]

Parking Line. The line separating parking and sidewalk. [*Distric Columbia.*]

Public Building. Every building used as church, chapel, or other plac public worship; also every building used as a college, school, public hall, hosp theater, public concert-room, public ball-room, public lecture-room, or for public assemblage. [*Boston, Chicago, Cincinnati, Cleveland, Denver Ka City, and Minneapolis.*]

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[*] Staying apartments. [*Kansas City.*]

[†] To be used jointly by separate buildings. [*Cincinnati and Cleveland.*]

buildings as shall be owned and occupied for public purposes for this he United States, the corporation of the City of Brooklyn, or other public within said city. [*Brooklyn.*]

ic Hall. Every theater, opera-house, hall, church, school, or other build-nded to be used for public assemblage. [*Milwaukee and Louisville.*]

rn Wall. No wall subdividing any building shall be deemed a **return** before mentioned, unless it is two-thirds the height of the external or alls. [*Cincinnati and Cleveland.*]

. A skeleton structure for storage or shelter. [*District of Columbia.*]
structure, enclosed only on one side and end, and erected on the ground. ancisco.]
or closed board structure. [*Denver.*]

-window. A store-window in which goods are displayed for sale or sement. [*District of Columbia and Denver.*]

re thereof. The square or level of the walls before commencing the r roof. [*District of Columbia.*]

dard Depth for Foundations. For brick and stone buildings, 14′ urb line. [*San Francisco.*]

dard Depth of Cellars. 16′, measured down from sidewalk grade at y line. [*Memphis.*]

dard Iron Door. Made of No. 12 plate-iron, frame or continuous x ⅜″ angle-iron, firmly riveted. Two panel doors, to have proper cross-ne panel on either side, fastened together with hooks or proper bolts top ttom, and with not less than two lever-bars. All doors hung on iron of ⅜″ x 4″ iron, securely bolted together through wall, swung on three fitting close to frame all around; sill between doors, iron, brick, or stone, not less than two (2) inches above floor on each side of opening. Lintel or, brick, iron, or stone. Floors of basement, when doors are to swing, r cement, in no case wood. [*Denver.*]

dard Skylight. Constructed of wrought-iron frames, with hammered -light glass not less than ½″ thick; not larger than 10′ by 12′, except by permission of the Inspector. [*Denver.*]

ehouse. (See **Warehouse Class.**)

et. All streets, avenues, and public alleys. [*Minneapolis.*]

ement-house. A building which, or any portion of which, is to be occu-r is occupied, as a dwelling by more than three* families living independ-f one another, and doing their cooking upon the premises. [*Boston, , and Kansas City.*]

y more than two families† above the second floor, so living and cooking. *and Kansas City.*]

ling which shall contain more than two rooms in front on each floor, or shall be built with a passage or arched way between distinct parts of the uilding, or which building shall be intended for the separate accommoda-different families or occupants. [*Charleston.*]

ater. Public hall containing movable scenery or fixed scenery which is de of metal, plaster, or other incombustible material. [*Chicago, Louis-d Milwaukee.*]

---

o instead of three. [*District of Columbia and Minneapolis.*]
on one floor, *but having a common right in the halls, stairways, yards, etc.* [*Provi-*

Thickness of a Wall. The minimum
Cincinnati, Cleveland, Kansas City, Milwau

Tinned Covered Fire-door. Wood d
wood, cross or diagonal construction, cove
sheet-tin, joints securely clinched and nail

Tower Projection. A projection desi
for ornamental windows, or for buttresse

Vault. An underground construction
of Columbia.]

Veneered Building. Frame structu
4′ wall of brick, instead of clapboard
Milwaukee, and Minneapolis, but not de

Warehouse Class. Buildings used
tories in which machinery is operated
and St. Louis.]

Width of buildings shall be compu
lengthwise of the beams shall be consi
building. [*New York and San Franc*

Wholesale store, or storehouse, s
to be used) exclusively for purpose
[*Chicago, Louisville, and Milwaukee.*

Wooden Building. A wooden
and Minneapolis.]

Any building of which an extern
part of wood. [*Denver and Distric*

Having more wood on the outsi
frames, doors, shutters, sash porti
or sheds, although the sides and e
iron or other metal, shall be deem
ton and Nashville.]

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* As applied to solid
† Or veneered. [*Mi*

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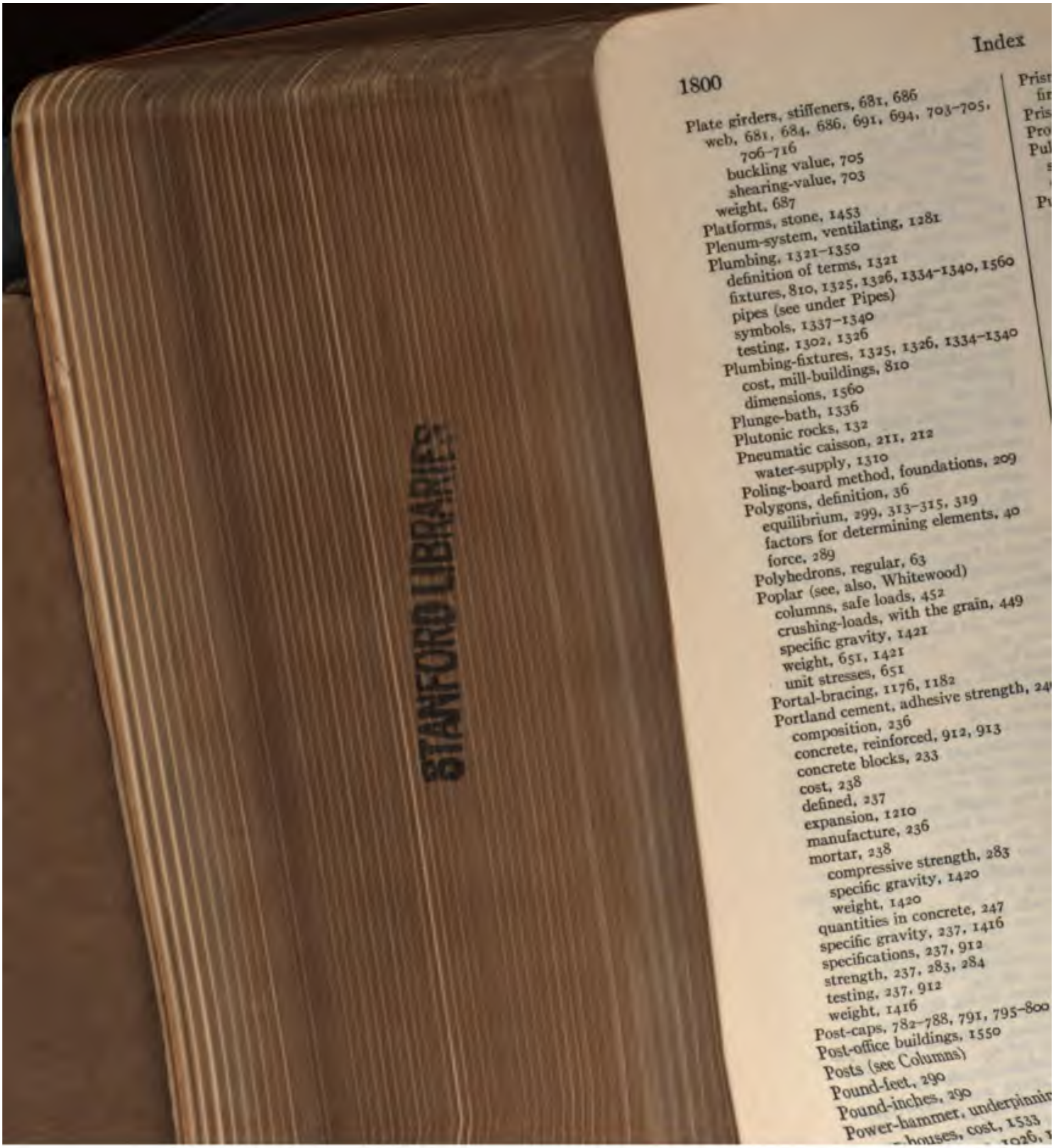

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